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ANALYSIS OF DATA FROM INSTRUMENTATION PROGRAM: LOCK AND DAM NO. 1 RED RIVER WATERWAY, LOUISIANA

by

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and

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US Army Corps of Engineers

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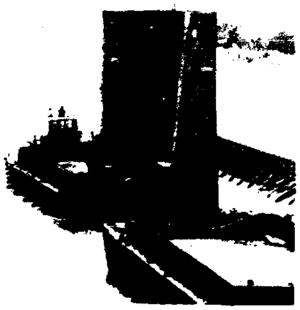

September 1989

Final Report

Prepared for US Army Engineer District, Vicksburg
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19. ABSTRACT (Continue on reverse if necessary and identify by block number)

The instrumentation program consisted of monitoring performance of the structure during and after the construction stage and during normal operation. Special instrumentation was concentrated at the gated dam pier, typical lock chamber monolith, and gate bay monoliths to evaluate the structure behavior such as bending moments, soil and uplift pressures, and structure deflections. In general, observed settlements were less than predicted settlements with observed base pressures along the base slab higher than predicted values. Deflections based on moment calculations and on strain data show good agreement, and moments calculated using uniform base pressure distribution were approximately equal to moments calculated based on strain data.

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PREFACE

This report presents the analysis of the instrumentation results in the report "Instrumentation Data Summary, Lock and Dam No. 1", November 1987. Both reports were sponsored by the US Army Engineer District, Vicksburg (LMK) and prepared by the US Army Engineer Waterways Experiment Station (WES).

The data and information for this report were compiled during and after construction of Lock and Dam No. 1; however, some instrumentation was destroyed before any measurements could be taken and some malfunctioned. Data reduction did not start until 1982.

Messrs. John M. Andersen, Roy E. Leach, Earl V. Edris, Engineering Group (EG), Soil Mechanics Division (SMD), Geotechnical Laboratory (GL), Dr. Victor H. Torrey III, Research Group, SMD, GL, and Mr. Bradford Walker Jr., (LMK) contributed valuable advice to the project. Mr. Chester Schneider (IPA) and Mr. William L. Hanks, EG, SMD, GL, assisted in the preparation of the plots. This report was prepared by Ms. Wipawi Vanadit-Ellis, EG, SMD, GL, Messrs. Robert L. Hall and Paul W. Graham, Structural Analysis Group, Structural Mechanics Division, Structures Laboratory.

Overall direction at WES was provided by Mr. G. Britt Mitchell, Chief, EG, SMD, and Dr. W.F. Marcuson III, Chief, GL.

COL Larry B. Fulton, EN, was Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurements used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
kips (force) per inch	175.1268	kilonewtons per metre
miles (US statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic inch	27.6799	grams per cubic centimetre
square feet	0.09290304	square metres
square inches	6.4516	square centimetres
square yards	0.8361274	square metres

PART I: INTRODUCTION

Purpose of Instrumentation Program

1. Lock and Dam No. 1 instrumentation was designed to monitor the performance during and after the construction phase and during normal operation. The various loading conditions, along with the consolidation of the soils, will affect the soil structure interaction. Installation of special instrumentation was concentrated at the gated dam pier, typical lock chamber monolith and gate bay monoliths to evaluate the structure behavior such as bending moments, soil and uplift pressures, and structure deflections. These measurements will be used to verify design assumptions.

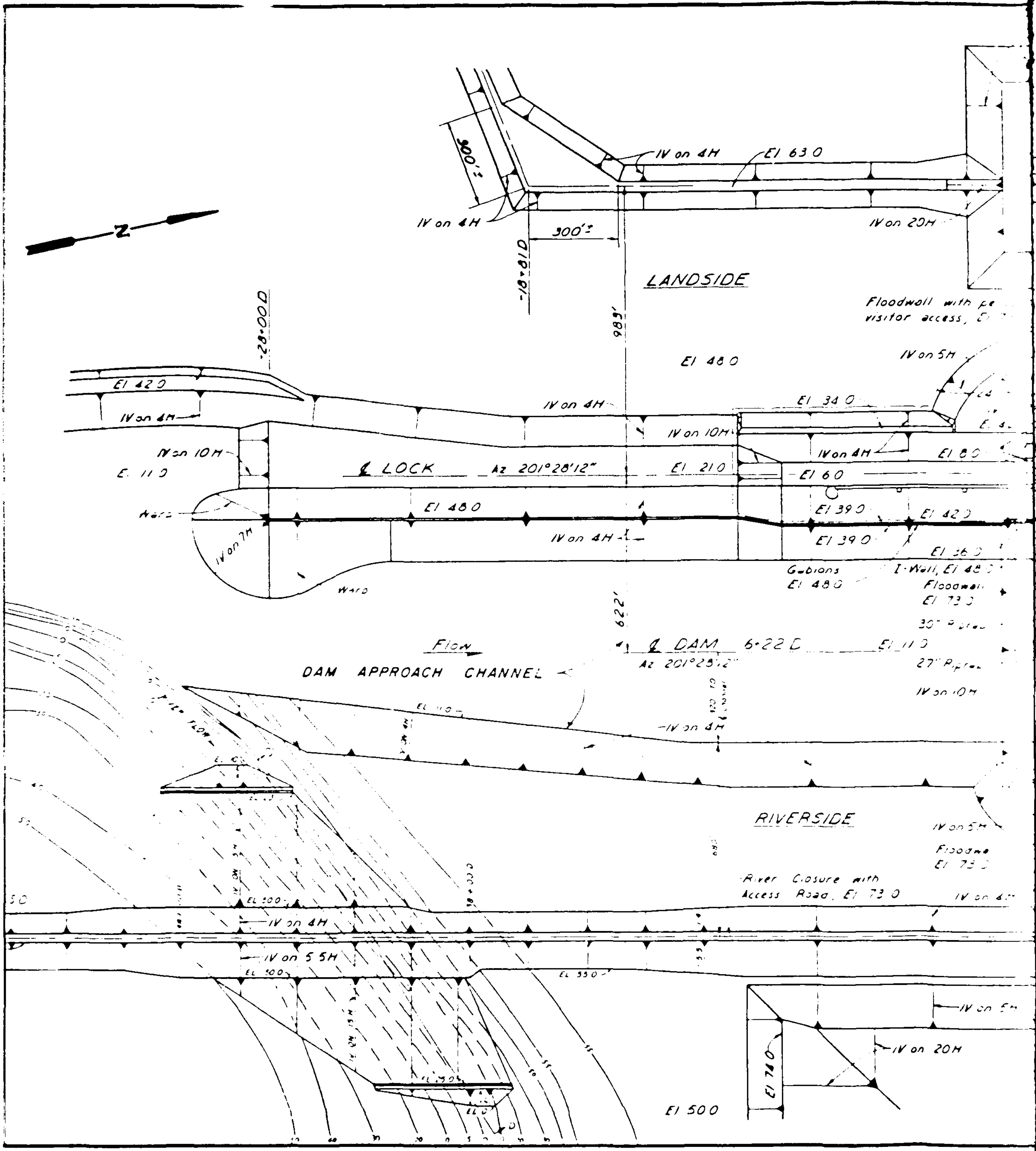
Scope of Report

2. This report represents the second phase in the study of the structural performance of Lock and Dam No. 1. The first phase was a summary of data and its reduction to tabular or graphical form and was submitted as "Instrumentation Data Summary, Lock and Dam No. 1," November 1987, hereinafter referred to as the "Data Summary". This second phase contains a summary of soil and foundation responses as observed from the instrumentation program. The analysis and evaluation of observations regarding the design were based on available data for the construction and operation phase.

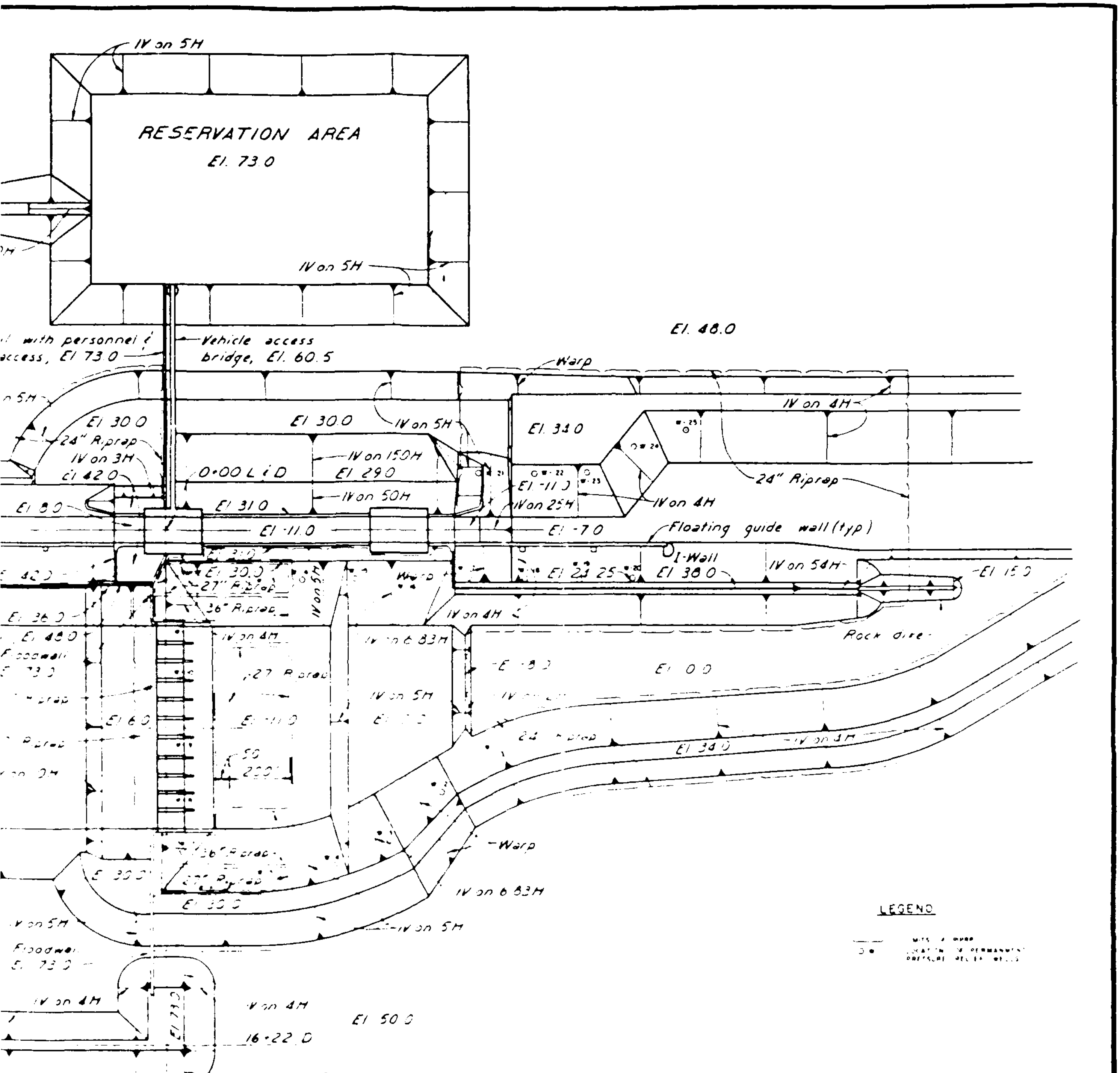
Description of Project

3. Lock and Dam No. 1 (shown on plate 1) is a part of the project "Red River Waterway: Louisiana, Texas, Arkansas and Oklahoma," and is the first in a series of locks and dams to provide navigation for barge traffic between the Mississippi River and Shreveport, LA. Lock and Dam No. 1 is in Catahoula Parish, LA, approximately five miles north of the settlement of Brouillete. Catahoula Parish is about 70 miles north of Baton Rouge, LA, and 16 miles west of the Mississippi River. The Lock and Dam No. 1 complex was built between river miles 42.6 and 50.5 of the Red River. At mile 50.5 the river was departing eastward from its original northerly direction to return to its

original direction at mile 42.6 forming a meander. The 1.7 mile long structure shortened the waterway by eight miles.



N
IV on 4H
El 63.0
900'±
IV on 4H
300'±
IV on 20H
18+81D
LANDSIDE
Floodwall with
visitor access
28+00D
985'
El 48.0
IV on 5H
El 42.0
IV on 4H
IV on 4H
El 34.0
IV on 10H
IV on 10H
IV on 4H
El 8.0
El 11.0
℄ LOCK
Az 201°28'12"
El 21.0
El 6.0
Ward
El 48.0
El 39.0
El 42.0
IV on 7H
IV on 4H
El 39.0
El 36.0
Gabions
El 48.0
I-Wall, El 48.0
Ward
El 73.0
622'
30" Riprap
Flow
℄ DAM
6+22D
El 11.0
DAM APPROACH CHANNEL
Az 201°28'12"
27" Riprap
IV on 10H
El 11.0
IV on 4H
RIVERSIDE
IV on 5H
El 73.0
River Closure with
Access Road, El 73.0
El 50.0
IV on 4H
IV on 5.5H
El 50.0
El 55.0
IV on 5H
IV on 20H
El 74.0
El 50.0



RED RIVER WATERWAY LA. TEX. ARK. OKLA.
MISSISSIPPI RIVER TO SHREVEPORT, LA.
DETAIL DESIGN MEMORANDUM NO. 9
SOILS DATA AND FOUNDATION DESIGN

LOCK AND DAM NO. 1

PLAN-LOCK, DAM, AND APPURTENANCES

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

JUNE 1977 FILE NO. H-2-27409

PLATE 1

PART II: HISTORY OF PROJECT

Design

4. General Design Memorandum No. 2 "Phase I, Plan Formulation and Site Selection" was prepared by the US Army Engineer District, New Orleans (LMN) and approved on 29 October 1976. General Design Memorandum No. 6 "Phase II, Project Design, Lock and Dam No. 1" was prepared by the US Army Engineer District, St. Louis (LMS) and approved on 3 August 1976. "Sources of Construction Materials, Lock and Dam No. 1" was provided by the LMN as Design Memorandum No. 8 and approved on 20 July 1987. "Soils Data and Foundation Design, Lock and Dam No. 1" was in the form of Detail Design Memorandum prepared by the LMN and approved on 1 November 1977. The LMN prepared a four volume Detail Design Memorandum #11. Volume I "Lock Masonry and Operating Equipment, Lock and Dam No. 1" was approved on 24 August 1979. Volume II, "Lock Gates and Operating Equipment, Lock and Dam No. 1", bears approval date of 16 May 1978. Volume III, "Masonry, Embedded Metals, Gates and Operating Equipment, Lock and Dam No. 1" was approved on 31 March 1978. The "Instrumentation Evaluation, Lock and Dam No. 1" representing the fourth volume of DDM No. 11 was prepared by the Waterways Experiment Station and approved on 7 June 1979.

Construction

General

5. Phases I, II and III construction contracts were awarded to Maharray-Houston Co., Memphis, TN, Arundel Corporation, Baltimore, MD and J.A. Jones Construction Co., Charlotte, NC respectively. Phase I consisted primarily of earthwork construction including approximately 3.5 million cubic yards of hydraulic excavation, 2.5 million cubic yards of embankment, the installation of CMP pipe culverts, access road and other miscellaneous appurtenances. Phase I was completed on 22 September 1978. Phase II consisted of the installation, operation and maintenance of the three dewatering systems, the installation of piezometers, testing of "H" piling, approximately 3.5 million cubic yards of structural excavation and other secondary roadway and embankment construction.

Phase II was completed on 31 January 1980. Phase III consisted of the remainder of the construction including excavation and embankment, "H" and sheet piling, baffle blocks, the lock and dam structures, stone riprap, installation of mechanical features, a maintenance and administrative building, roads, and other miscellaneous work required for the construction of a complete and operational lock and dam. Phase III was completed on 6 December 1985.

Monitoring

6. During construction, monitoring of the instruments was performed by technicians working for Law Engineering Consulting Engineers under direct contract to the U.S. Army Engineer District, New Orleans.

7. Monitoring of the settlement process was more complex. This was done by contract surveyors under direct contract to the New Orleans and Vicksburg Districts and the contract for surveying was awarded on a yearly basis, but not necessarily to the same contractor.

8. The transfer of the project from the New Orleans to the Vicksburg District took place in November of 1985. After this date the monitoring of the piezometers and strain gauges was performed by the Instrumentation Services Division of the Waterways Experiment Station.

Reading Schedule

9. Piezometers in the lock were read once a month when the differential head (the upper pool minus the lower pool) was less than five feet and twice a month if the differential head was five to ten feet. Upper and lower pool levels were recorded when piezometers were read.

10. Piezometers in the dam were read on the same schedule as the lock. Settlement plates in the lock chamber and behind the lock walls were subjected to observation at 6-month intervals in the first year after completion of construction and then at one year intervals.

11. Reference bolts were monitored during the first year at 3-month intervals and at yearly intervals afterward.

12. All settlement observations were referenced to permanent bench marks. The distance between reference bolts was scheduled to be read by scale

to the nearest .01 foot. The measurements were taken at the same time as the measurements of the upper pool. The distance between the landside and riverside walls of the lock was be measured between reference bolts at the center of an interior monolith. Only three chaining measurements across the lock chambers were taken, one at monolith L-10 and two at L-14, on 19 September 1985 and 4 September 1986. On surface monuments (stainless steel reference plates), observations were made at 3-month intervals in the first year and afterwards semiannually for two years and before, during, and after maximum differential heads on the structure.

13. Resteel strain gages were read monthly for the first two years and semiannually afterwards. Soil pressure meters were read monthly for the first two years and semiannually afterwards. Pile and sheet pile strain gages were read monthly for the first two years and semiannually afterwards.

Damaged Instruments

14. This report excludes rebound analysis since all heave points were lost.

15. Several temporary settlement points, stainless steel plates, and settlement plates were destroyed. Insufficient data and unreported events for the existing instruments made settlement analyses difficult. Figures 1 and 2 show the result of a mud cleaning operation in the lock chamber (monolith L-2), where several settlement plates were scraped up by a front end loader.

Incorrect Survey Elevation

16. In 1982 the National Geodetic Vertical Datum (NGVD) corrected elevations on the first order level line from Simmesport, LA. to Index, AK. This changed the elevations of the bench marks from 45.380 feet to 44.970 feet. Lock and Dam No. 1 was built 0.410 feet lower than design. Since the construction elevation was based on bench marks prior to the survey correction, this report reflects data 0.410 feet higher than actual.

Figure 1. Mud cleaning operation at lock monolith L-2.

Figure 2. A settlement plate scraped up by a front-end loader during a gate repair operation.

Part III: FOUNDATION DESIGN

Foundation Conditions

Site Conditions

17. The project is located within the Gulf Coastal Plain Province near the western edge of the Mississippi River Flood Plain. The immediate area is characterized by natural levees, backswamps, oxbow lakes, and the ridge and swale topography of point bar accretion deposits. The natural levee, approximately 10 - 30 feet thick, provides local relief of about ten feet above the surrounding point bar and swamps. This deposit consists of soft to very stiff clays and silts which were deposited in the area during high water stages when the Red River topped its banks. The eastern section of the site is composed of point bar deposits of the main channel of the Mississippi River. These deposits consist of massive silts with layers of fine sand, silty sand, and clay. The central portion of the site is more uniform with backswamp clay overlying the substratum sands. These massive deposits consist of medium to very stiff homogeneous clays with few small lenses of silt, silty sand, and sand. The substratum sand layer comprises the base of the overburden at the site. It exists from elevation -50.0 feet to elevation -150.0 feet. Small lenses of clay, silt, and silty sand as well as areas of gravel exist in the sand layer. The western portion of the site has been traversed by tributaries of the Mississippi or Red Rivers. This resulted in the partial to complete removal of the backswamp material found in the central portion. In its place is a heterogeneous sequence of tributary point bar and backswamp deposits which in some places lie directly on substratum sands and in others on remnants of the massive Mississippi River backswamp.

Field investigations

18. Subsurface borings and groundwater studies were the principal sources of data used in the investigation of the ground conditions at the project site. A drilling program consisting of 139 borings was initiated by the LMN. The undisturbed borings were made using a 5 inch steel tube piston type sampler except in thick granular deposits where samples were obtained by using

a 1-3/8 inch inside diameter splitspoon sampler using a 140 pound driving hammer and a 30 inch drop where zones of coarse sands and gravels were encountered. The general type borings were taken with a 1-7/8 inch diameter core barrel sampler in the cohesive soil. The samples were subjected to tests in the field and to laboratory tests in the New Orleans District Soils and Materials Laboratory and in the Geotechnical Laboratory at the Waterways Experiment Station.

Laboratory tests

19. The overall testing program was carried out to interpret the geologic environments existing at the site and to develop design criteria. Laboratory tests consisted of water content determinations and grain size analyses (sieve, hydrometer, and D_{10} size) on selected samples of sand and silt. Liquid and plastic limits and unconfined compression tests (UC) were made on selected samples of cohesive soils. A series of shear tests were performed on selected undisturbed samples for design purposes. Unconsolidated undrained (Q) triaxial compression tests were made on selected samples of cohesive soils. Consolidated undrained (R) triaxial compression tests were made on selected samples of clay, silt, silty sand, and sand. Consolidated drained (S) direct shear tests were performed on selected samples of clay, silty sand, and sand. Consolidation (C) tests, including compression, rebound, and recompression cycles were performed on selected soil samples for use in settlement and rebound analyses. The results of the tests made are presented and discussed in detail in the "Lock and Dam No. 1, Detail Design Phase II, Soils Data and Foundation Design, Design Memorandum No. 9," dated 15 July 1977. The design parameters from these tests are shown on plate 2.

Groundwater studies

20. Groundwater studies were also conducted at the Lock and Dam No. 1 project site prior to the beginning of construction. The water level in each boring was recorded during the boring program. Four piezometers were installed and observed by the LMN. The United States Geological Survey (USGS) also installed three observation wells at the site. The data from the wells and piezometers indicate the groundwater levels vary with changes in the Red River and also reflect seasonal changes. The piezometer readings indicate that while the hydrostatic head in the substratum sands reacts with the stages in the Red River, the level of the head is slightly less than the river stages

and a two to four week time lag is experienced between stages in the river and corresponding hydrostatic head in the substratum sands. The shallow piezometers and observation wells installed in the fine grained overburden indicate a groundwater level which varies from the ground surface to about 10 feet below the ground. Detailed results of the groundwater studies are presented in "Agricultural Observation and Groundwater Study, Red River Waterway Project, Appendix 1, Lock and Dam No. 1," submitted June 1975.

Design considerations

21. The design for Lock and Dam No. 1 was based on engineering manuals, previous reports, and design criteria used for the Arkansas River Project. Table 1 contains the factors of safety and soil parameters used for design.

22. The principal elements of design for a U-frame section consist of ensuring the safety of the chamber sections against floatation, horizontal sliding, settlement and shear failure into the dam outlet channel. The stability of the dam was investigated against horizontal sliding, overturning, and floatation. The construction, dewatered and operating conditions were investigated for both the lock and dam. Detailed analyses are presented in "Lock and Dam No. 1, Detail Design Phase 2, Soils Data and Foundation Design, Design Memorandum No. 9," published July 1977.

Settlement Analyses

23. Consolidation tests performed show that the clays in the natural levee deposits are normally consolidated and the backswamp clay deposits are slightly to moderately overconsolidated. Analysis of the settlement induced by embankment loading indicates maximum foundation settlement of one-half foot.

24. Lock settlement analyses were performed for instrumented chamber sections along the adjacent earthen backfill for the construction and operating cases. The lock sections and stratification used in the settlement analyses are shown in figures 3 through 5. The magnitude of settlement and rebound is shown on plate 3. Ultimate settlements were computed for the following cases.

a. Case I. Construction condition: structure complete with backfill in place.

b. Case II. Operating condition: upper pool at El. 40.0, lower pool at El. 4.0, chamber pool at El. 40.0.

Table 1

Soil Design Parameters and Factors of Safety

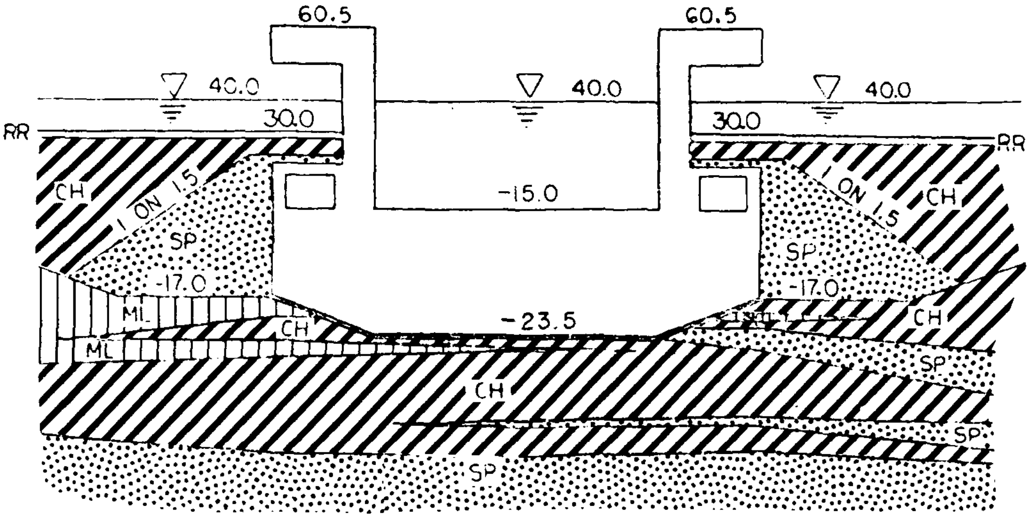
Structures:		
Sliding:	without piles	Q and S = 2.0
	Except for lock sliding upstream to downstream	Q = 2.0, S = 1.5
Sliding w/piles:	deep seated failure	Q and S = 1.3
		Earthquake (Seismic Coefficient = 0.05) = 1.0
Overturning:		Resultant within kern
Bearing Capacity:		Q and S = 2.0
Uplift:		= 1.25
Piles:		
Theoretical Equations:		Compression = 3.0, Tension = 3.0
Coefficients of Lateral Earth Pressure:		Compression = 1.0, Tension = 0.7
Load Tests:		Compression = 2.0, Tension = 1.75
Occasional Loads and Earthquake Loads:		Compression = 1.5, Tension = 1.75
Construction Excavation and Channel Slopes:		
Construction:		Q = 1.3
Operation:		
	Normal: Adjacent to structure:	Q = 1.5, S = 1.3
	Other:	Q and S = 1.3
	Steady Seepage and Sudden Drawdown:	S = 1.25
	Earthquake:	Q and S = 1.0 (Seismic Coefficient = 0.05)
Dewatering and Pressure Relief:		
Apply to Well Spacing:		FS = 1.15
Pressure Head Criteria:		No higher than 5' below bottom excavation

(Continued)

Table 1 (Concluded)

Lateral Earth Pressures Applied to U-Frame Structure:

Type of Soil	Design K
Sand	0.5
Clay	0.8



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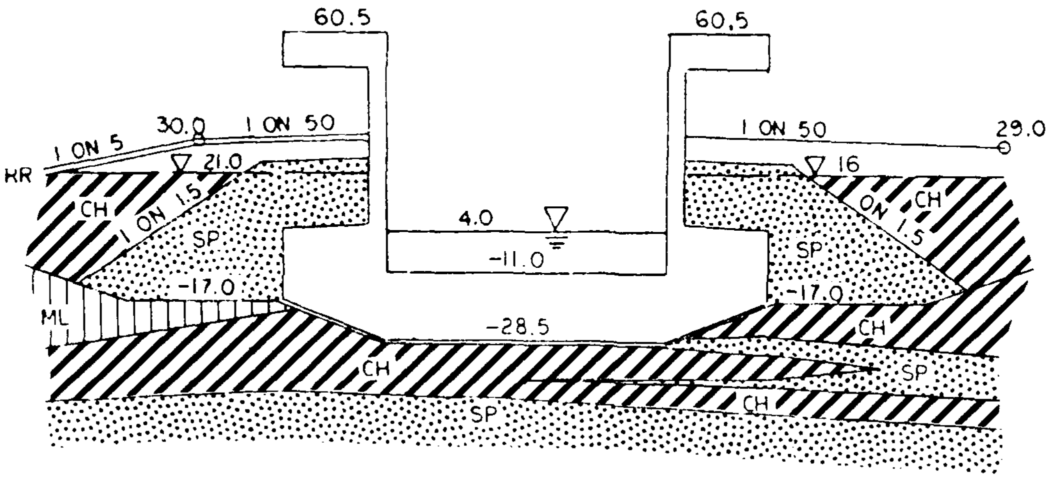
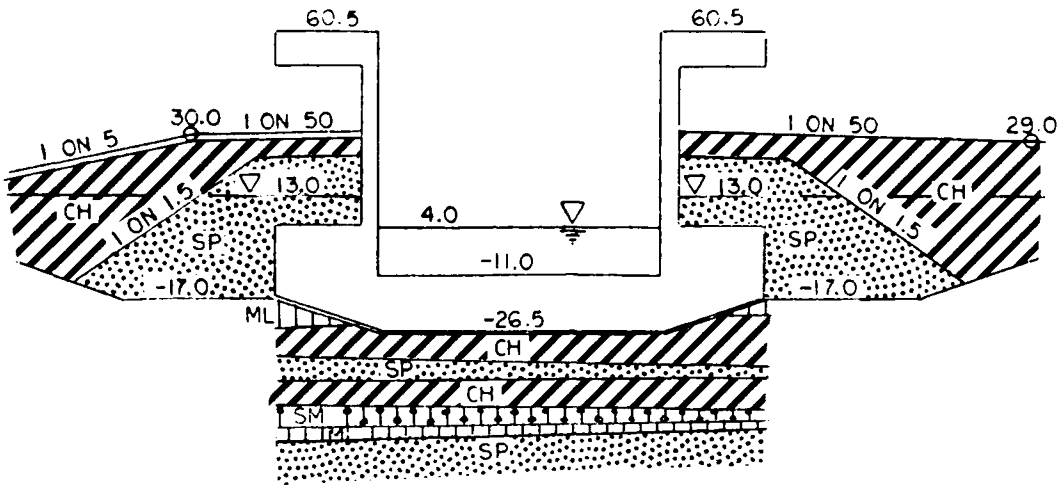


STA. 0+45 L

Figure 3. Stratification for lock monolith L-2.



STA. 6+87 L

Figure 4. Stratification for lock monolith L-16.

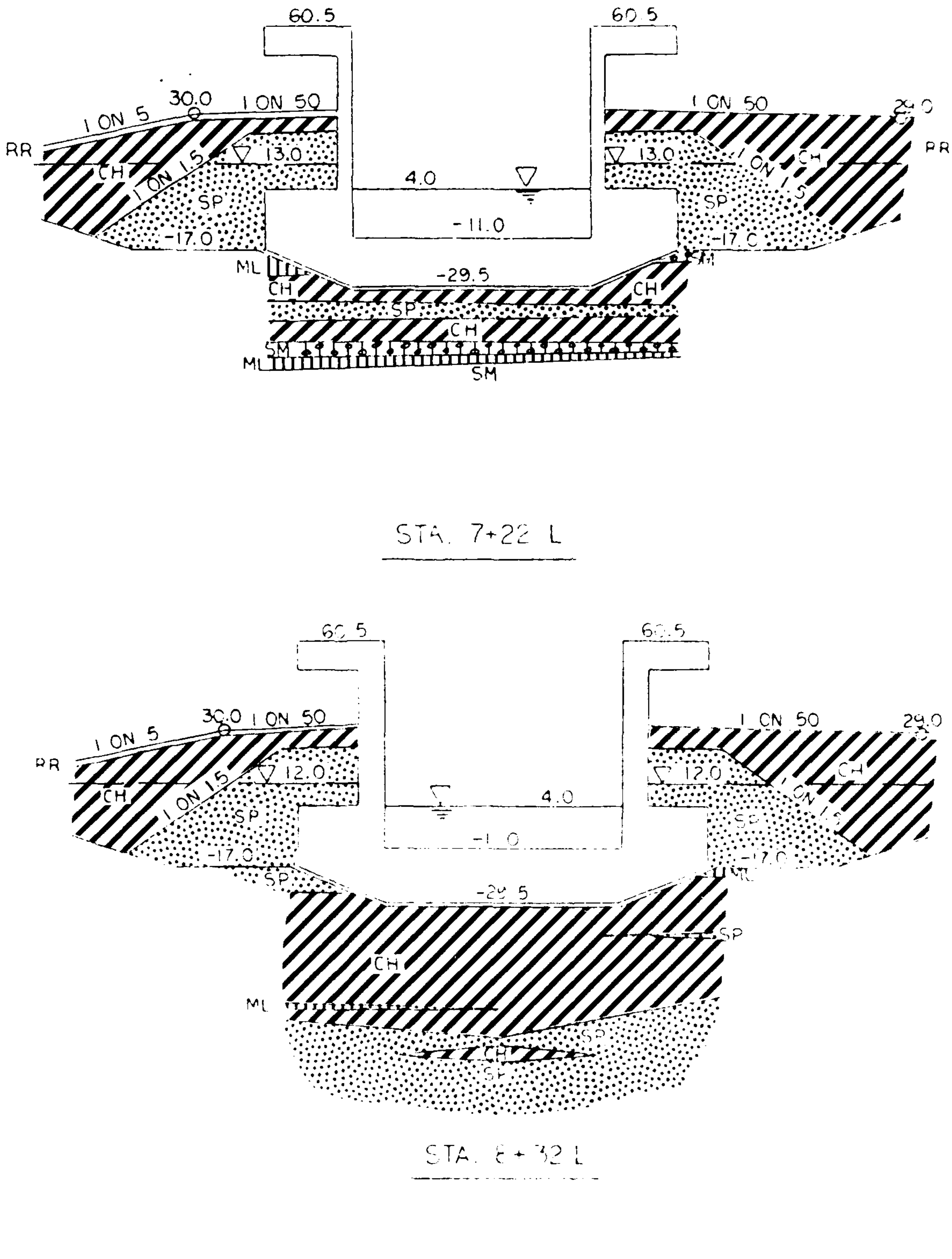
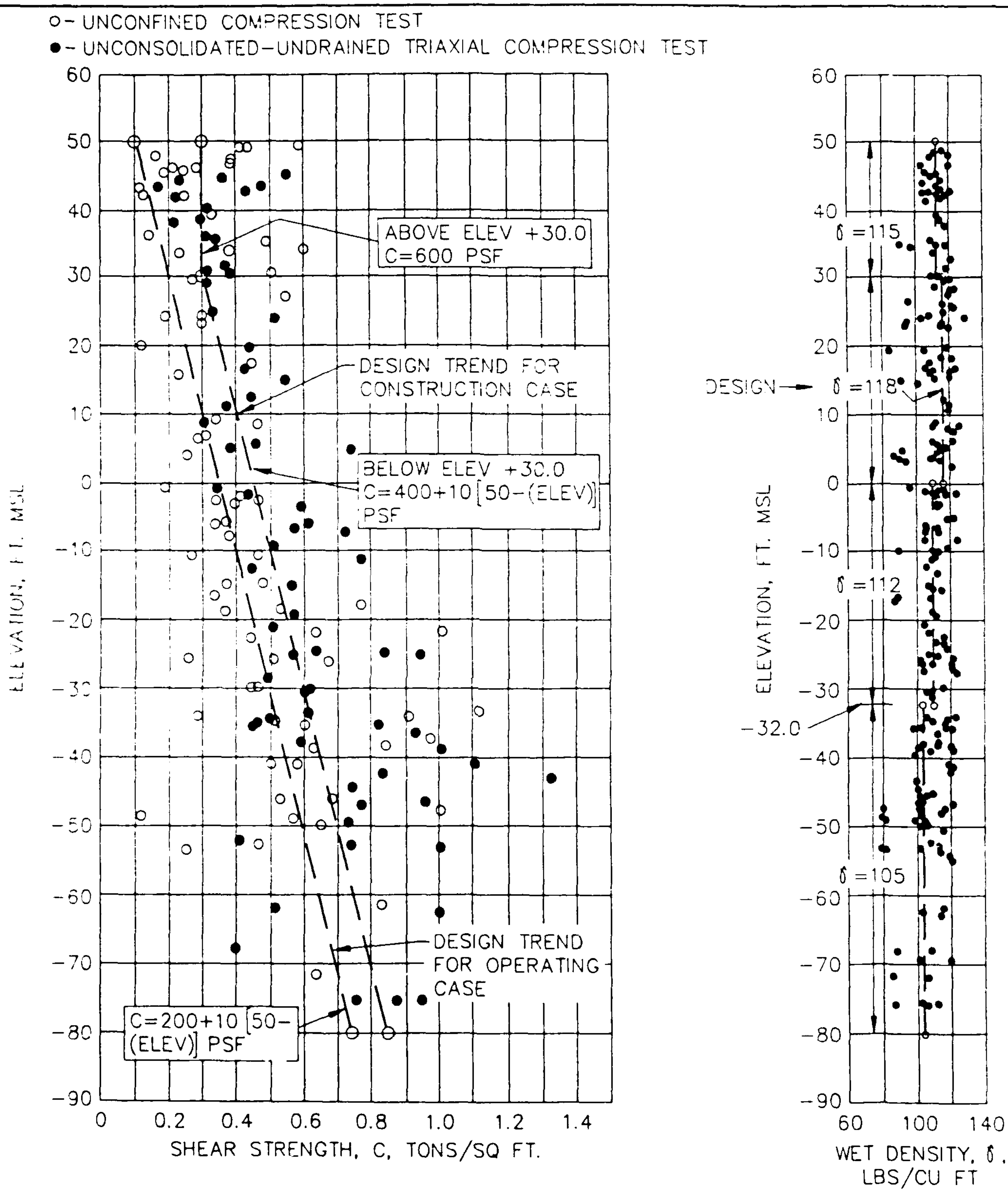
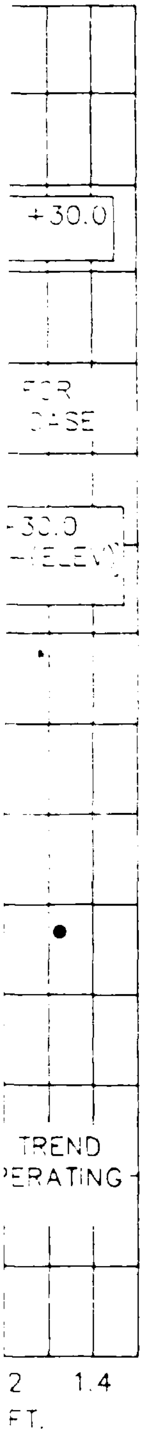


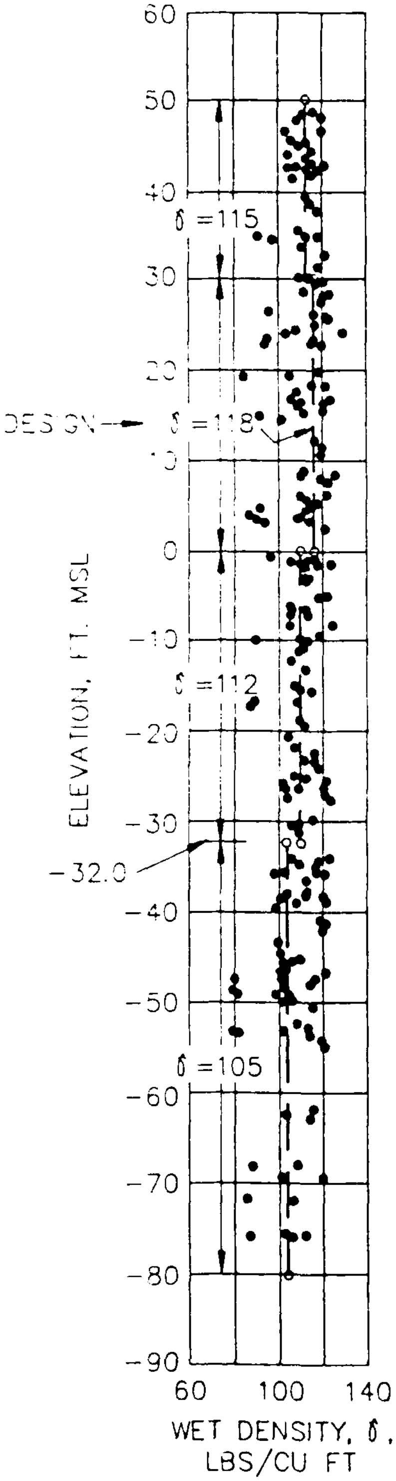
Figure 5. Stratification for lock monolith L-17.



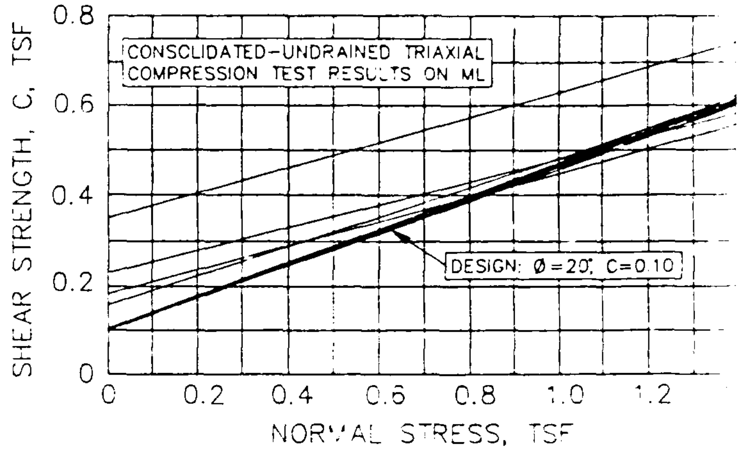
UNDRAINED SHEAR STRENGTH AND WET DENSITY DESIGN TRENDS
FOR NATURAL LEVEE AND BACKSWAMP CLAYS

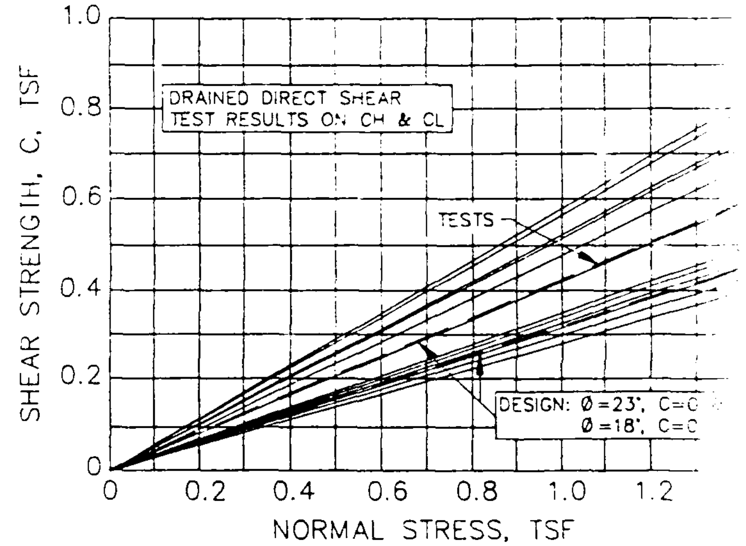
:OMPRESSION TEST

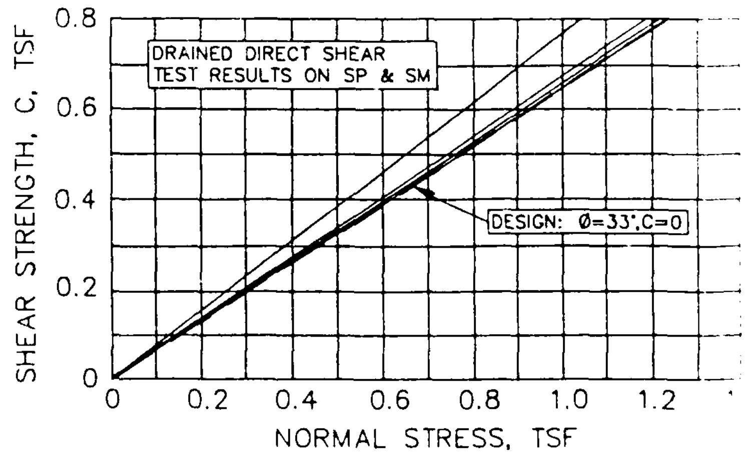


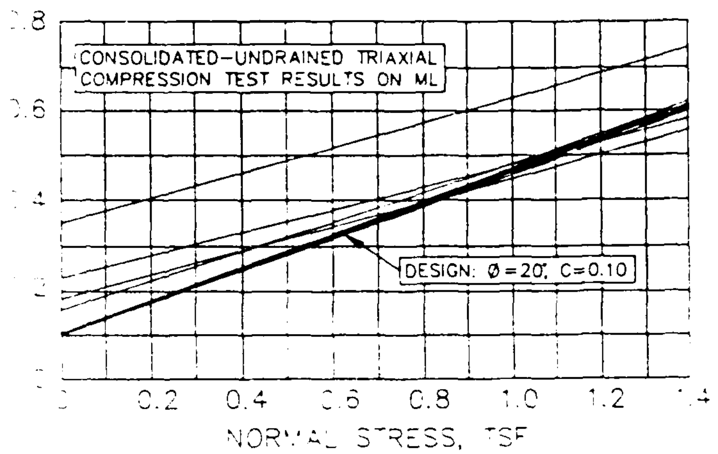


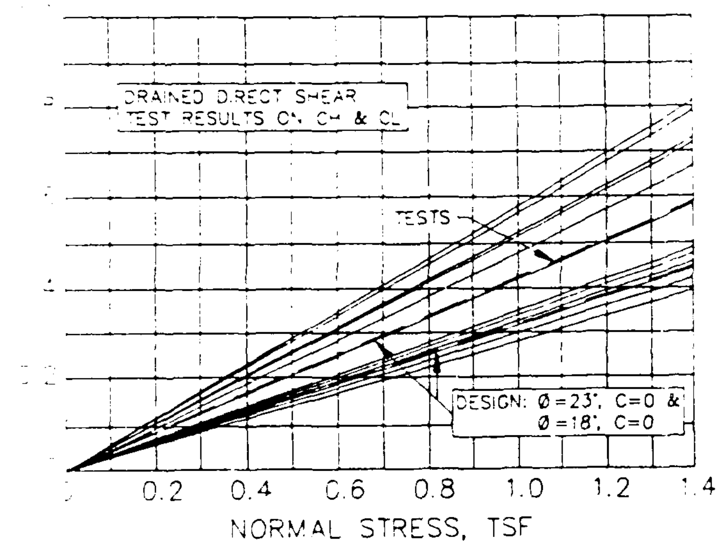
:D WET DENSITY DESIGN TRENDS
ND BACKSWAMP CLAYS

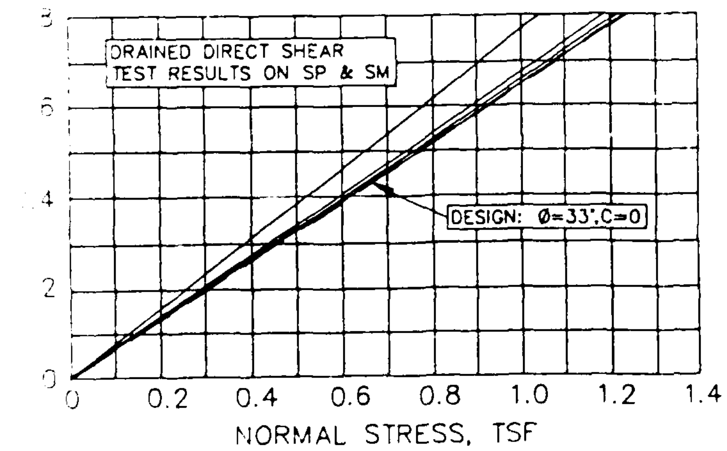












RED RIVER WATERWAY, LA., TEX., ARK. AND OKLA.
MISSISSIPPI RIVER TO SHREVEPORT, LA.
DETAIL DESIGN MEMORANDUM NO. 9
SOILS DATA FOUNDATION DESIGN

LOCK AND DAM NO. 1

DESIGN PARAMETERS

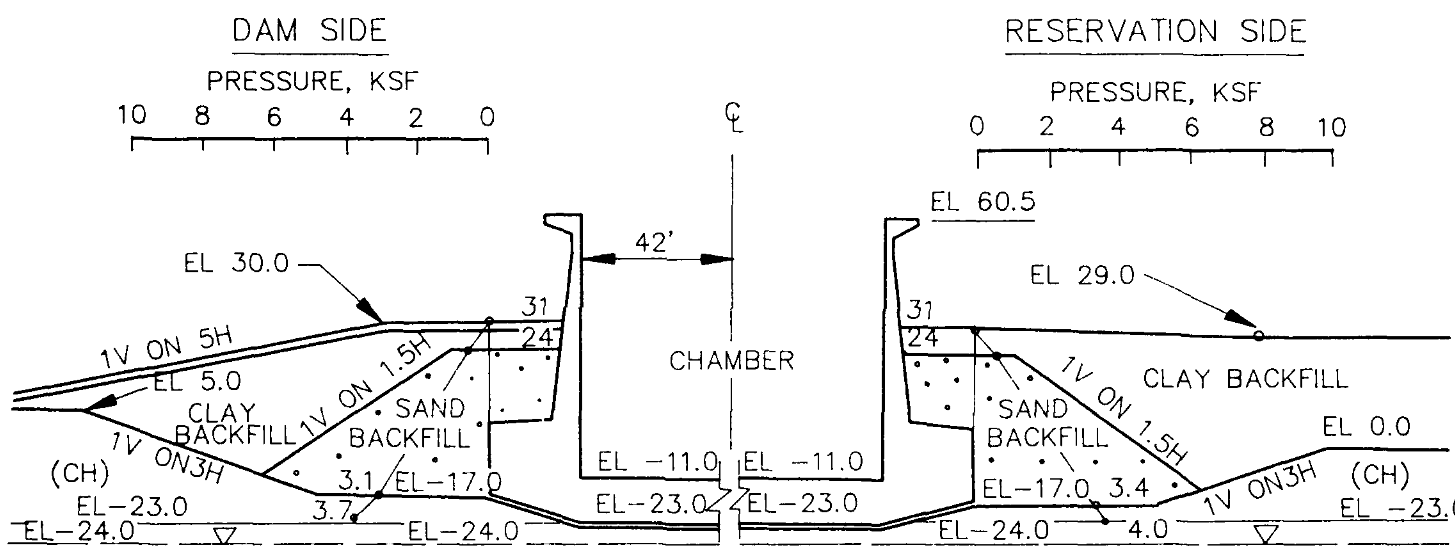
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JUNE 1977 FILE NO. 14-2-27409

Bearing capacity analyses

25. Base pressures and bearing capacity for the lock were determined for both the construction and operating cases. Ultimate bearing capacities were determined by Terzaghi's bearing capacity formulas. Design Memorandum No. 9, Lock and Dam No. 1 contains detailed analyses.

Lateral earth and water pressures

26. Lateral at-rest earth pressures on the walls are influenced by the type of backfill, method of placement, length of time in place, degree of shear strains imparted into the backfill during its useful life, effects of fluctuating water conditions, and seismic disturbances. For sand backfill, a lateral earth pressure coefficient of 0.5 was used for design. For clayey backfill, a lateral earth pressure coefficient of 0.8 was used for design. The lateral earth and water pressure diagrams were developed by a computerized analysis, using Coulomb's method of planes. The resulting diagrams used for analysis with various features under the construction, dewatered and operating design cases are shown in figures 6 through 8.



CASE I, CONSTRUCTION

Figure 6. Predicted earth pressures for the construction condition.

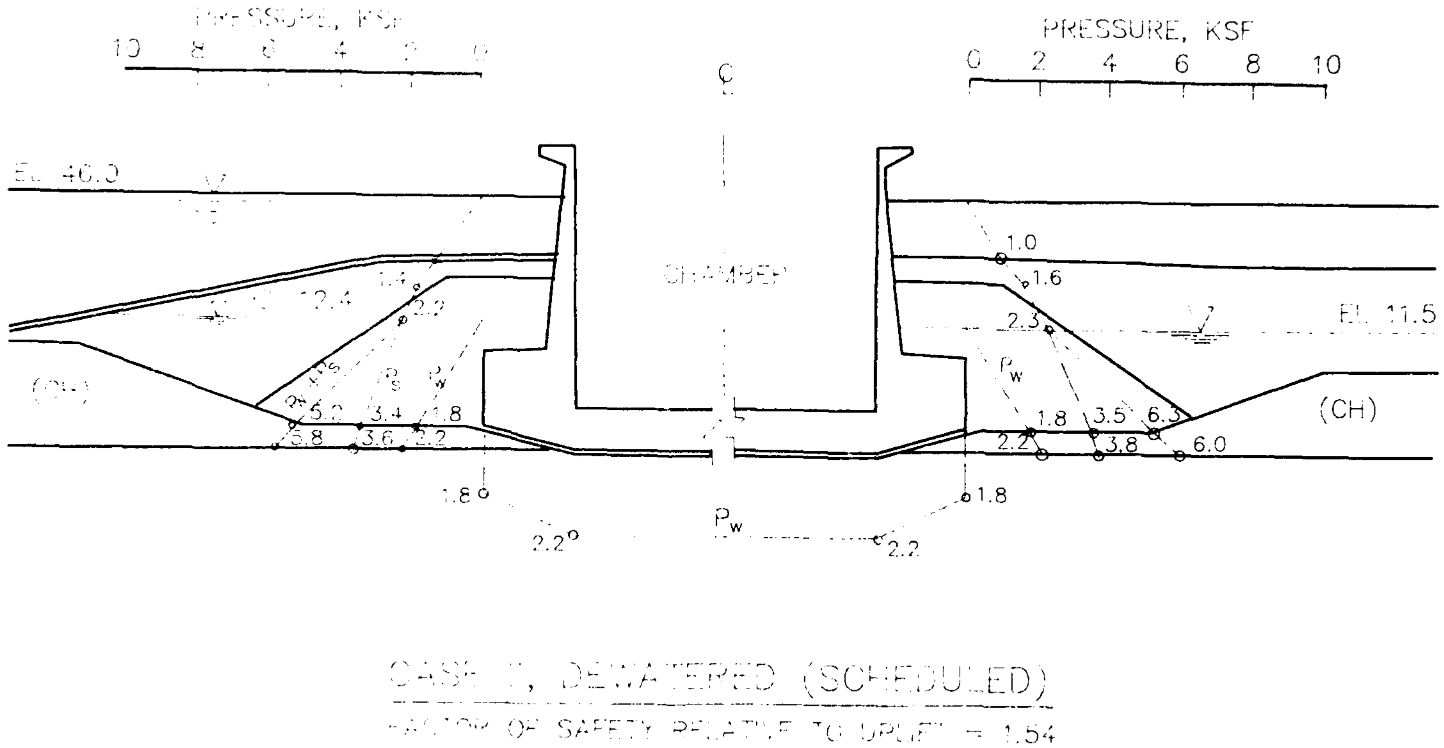
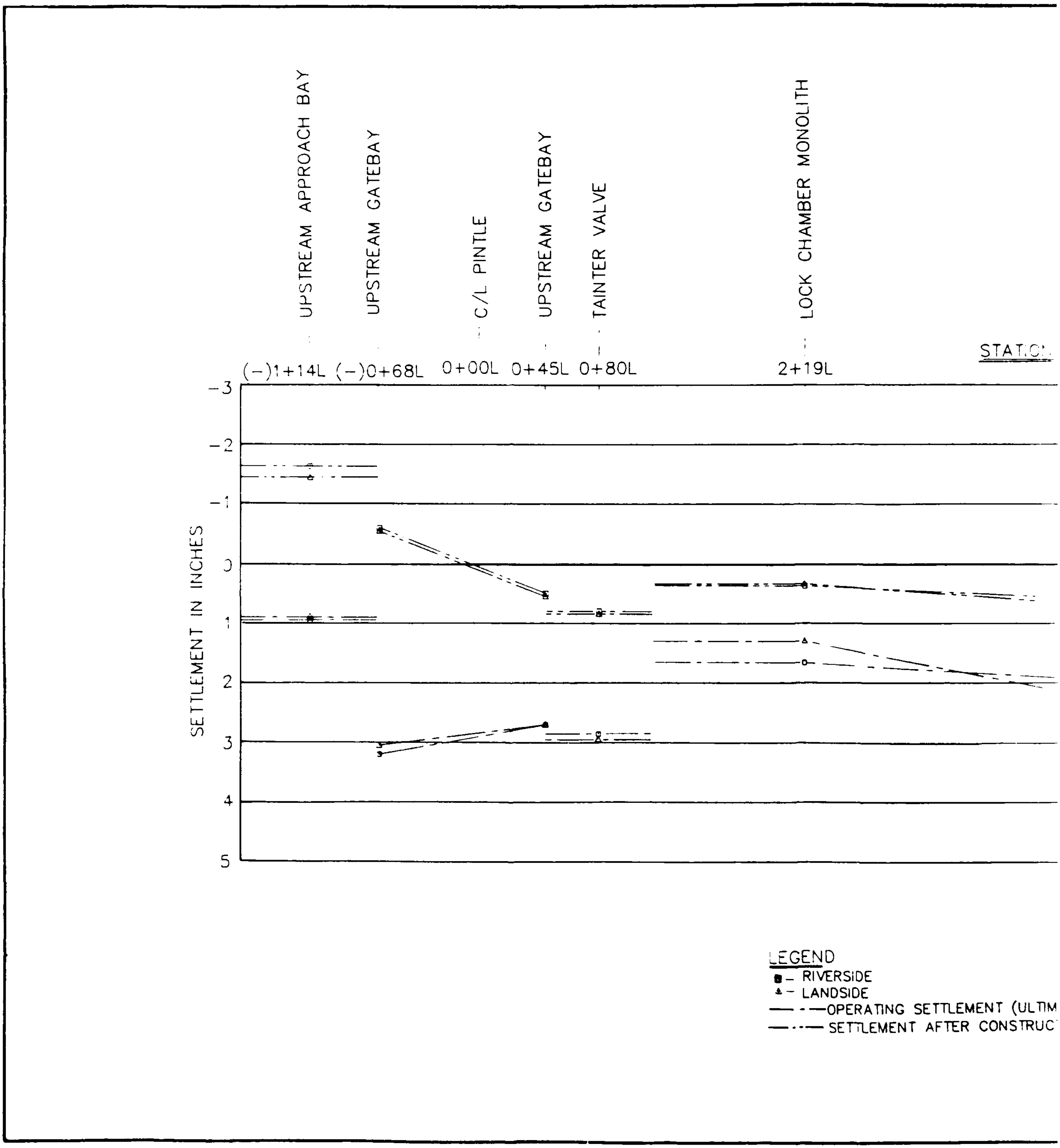
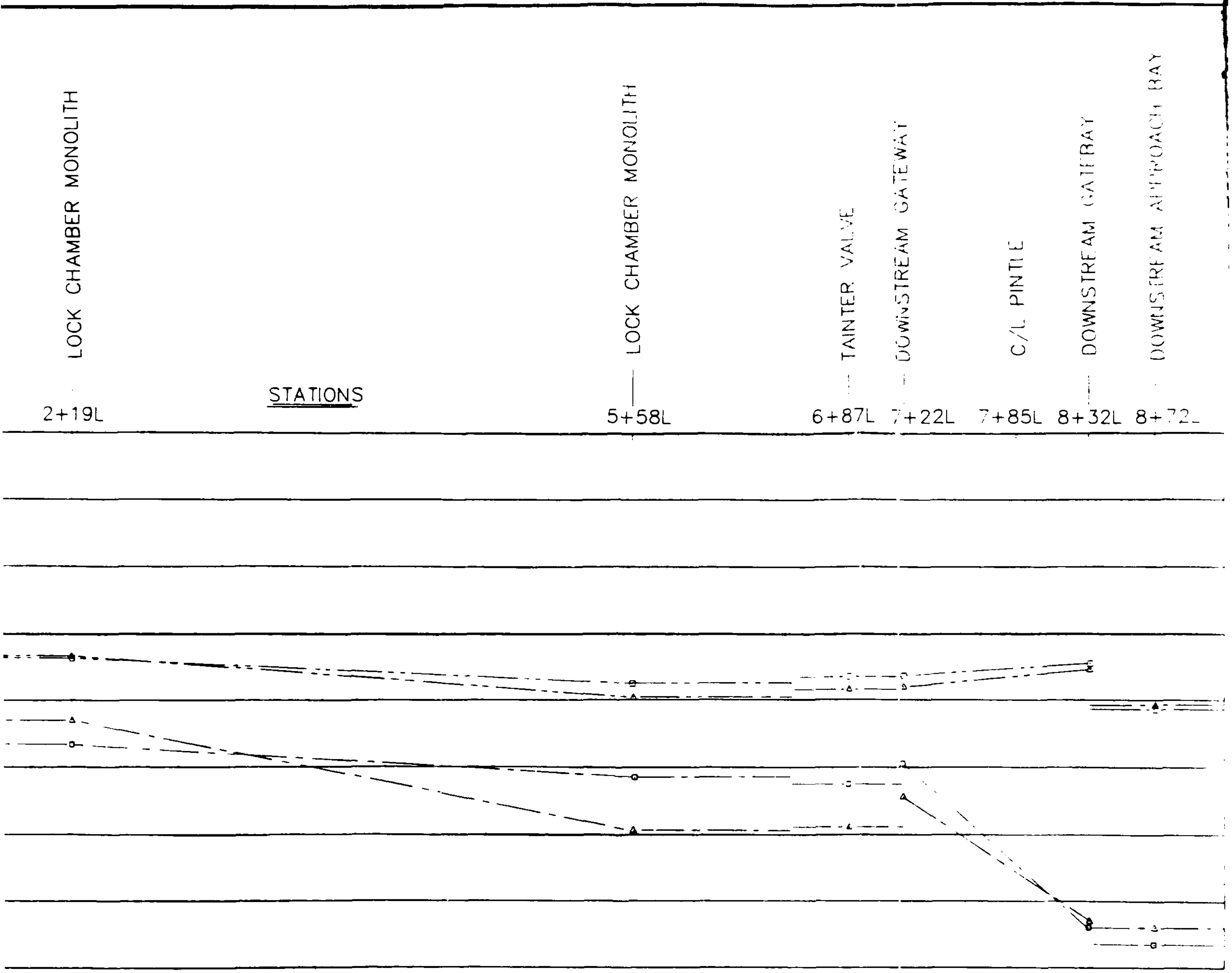


Figure 7. Predicted lateral and uplift pressures for the dewatered condition.

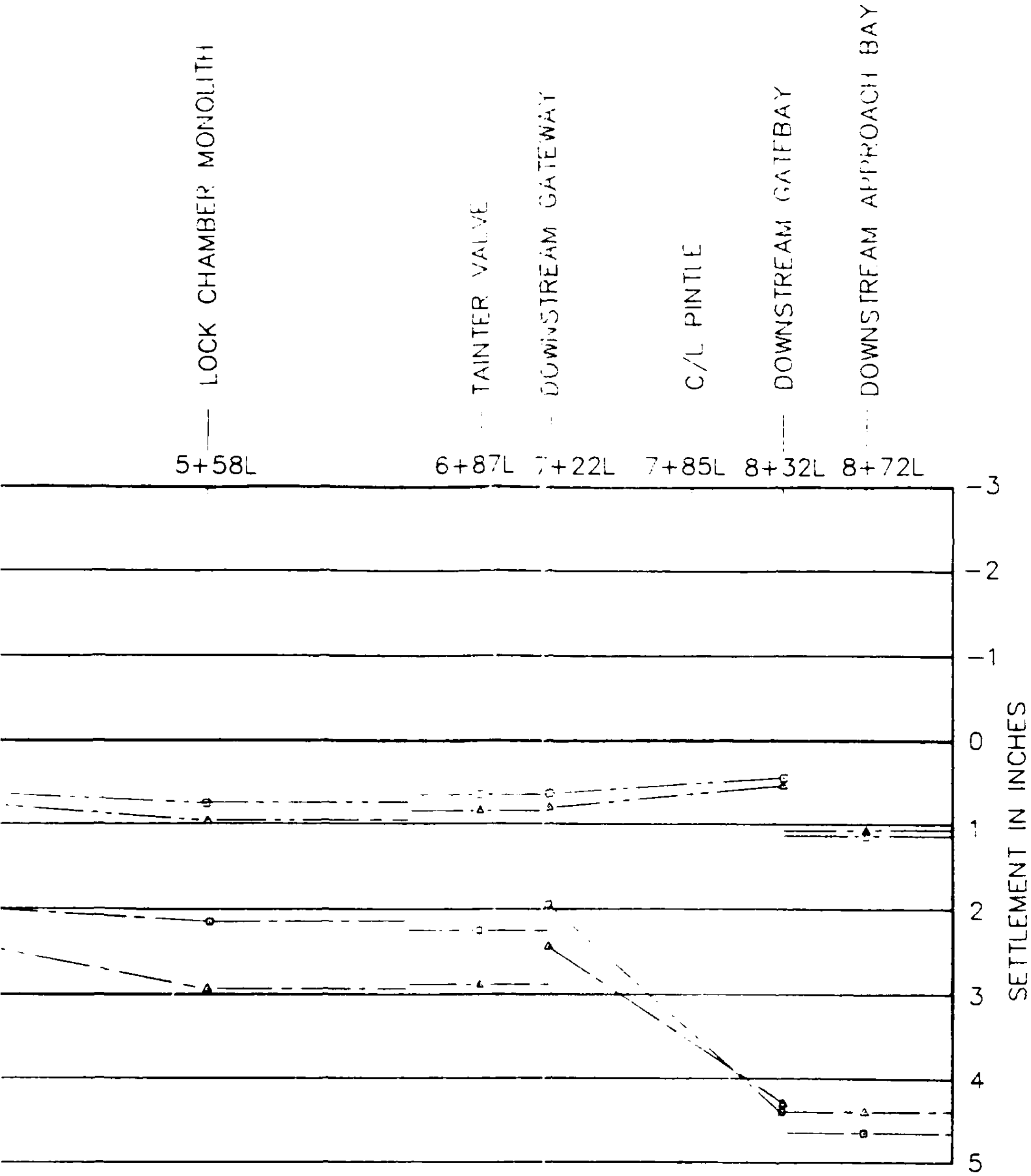

UPSTREAM APPROACH BAY
UPSTREAM GATEBAY
C/L PINTLE
UPSTREAM GATEBAY
TAINTER VALVE
LOCK CHAMBER MONOLITH
STATION
(−)1+14L
(−)0+68L
0+00L
0+45L
0+80L
2+19L
SETTLEMENT IN INCHES
−3
−2
−1
0
1
2
3
4
5
LEGEND
RIVERSIDE
LANDSIDE
OPERATING SETTLEMENT (ULTIM
SETTLEMENT AFTER CONSTRUC



LEGEND
- RIVERSIDE
- LANDSIDE
— - —OPERATING SETTLEMENT (ULTIMATE)
—·-·— SETTLEMENT AFTER CONSTRUCTION

ASSUMPTION: SETTLEMENT AFTER FIRST LIFT POUR. (6' CF CONCRETE)

JUN



ASSUMPTION: SETTLEMENT AFTER FIRST LIFT POUR. (6' CF CONCRETE)

RED RIVER WATERWAY, LA., TEX., ARK & OKLA.
MISSISSIPPI RIVER TO SHREVEPORT, LA
DETAIL DESIGN MEMORANDUM NO.9
SOILS DATA AND FOUNDATION DESIGN
LOCK AND DAM NO.1

LOCK SETTLEMENT ANALYSIS
SETTLEMENT OPERATING
(ULTIMATE) AND AFTER CONSTRUCTION

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

JUNE 1977

FILE NO. H-2-27409

PLATE 3

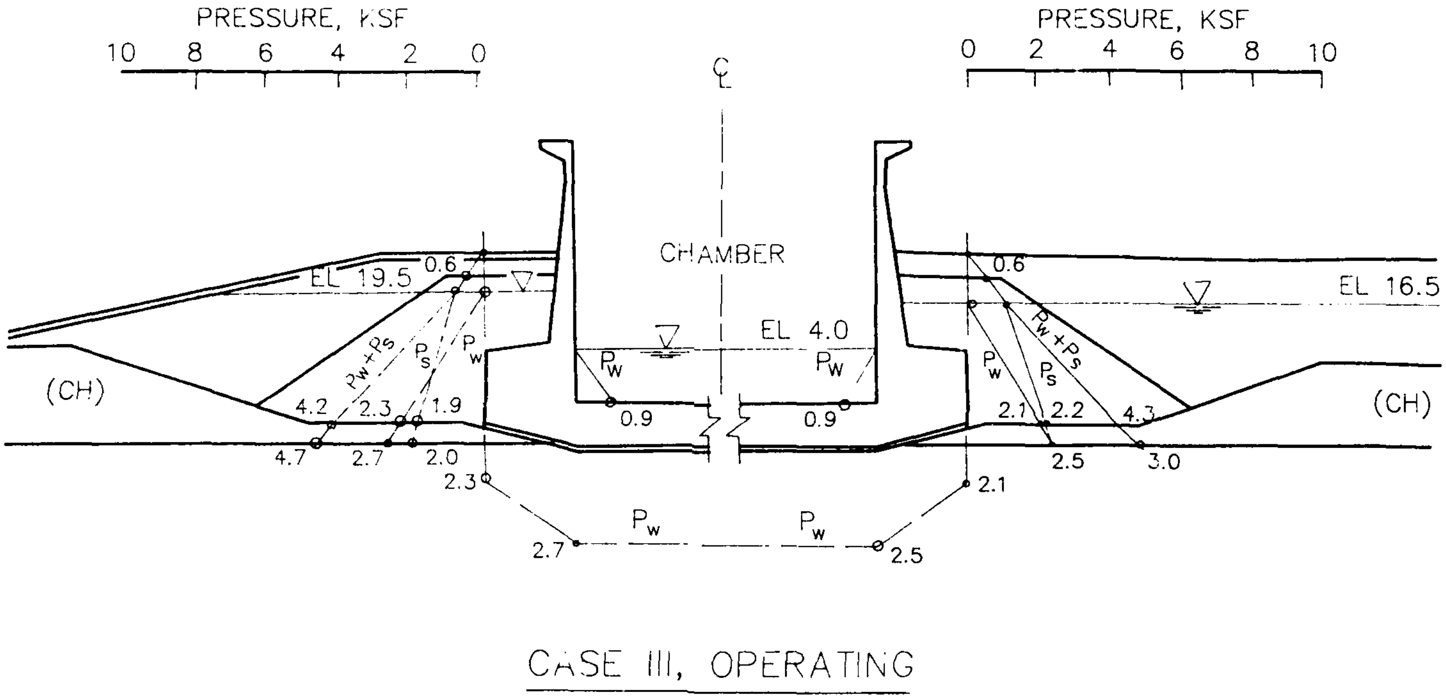


Figure 8. Predicted lateral and uplift pressures for the operating condition.

PART IV: TYPES OF INSTRUMENTS

Piezometers (P)

27. All the piezometers (except three) on this project are Casagrande open-tube piezometers. The readings are obtained by monitoring the water head in the piezometers directly. Open-tube piezometers are read by inserting an electrical probe through the riser pipe. The circuit is closed when the weighted probe contacts the water which manifests on the scale of ammeter. The depth to top of water in the pipe is read from the marking on the probe's cable. Most of the piezometers on this project are partially embedded in concrete. Piezometers installed in side walls of the lock are offset through nearly horizontal risers sloped 1 on 20. The sections of riser pipes were added during construction.

Surface Monuments (SM)

28. Two types of surface monuments are installed on this project. Type A surface monument measures movement of concrete structures while type B measures the surface movement of soil embankments. Surface monument type A consists of a shaft 1-1/4 inch in diameter and 18 inch long embedded in the concrete structure. Surface monument type B consists of a 4 foot pedestal supported by a steel plate embedded in the ground. The surface monument tube is identical in both cases.

Reference Bolts (RB)

29. Reference bolts consist of round head brass bolts, 3 inch long and 1/2 inch diameter, directly embedded in the concrete. Reference bolts provide information about horizontal and vertical displacements of the structures. The reference bolts are installed as a single bolt or in groups of four.

Strain Gages (SG)

30. For monitoring of bending moments, two sheet piles in the dam monolith are each instrumented with strain gages. Each of eleven elevations on these sheet piles has four groups of four strain gages. Each group of four gages is connected by means of four-arm Wheatstone bridge circuits requiring readings from four leads. A total of 24 H-piles are instrumented with strain gages. The strain gages are installed on the web and four corners of the flanges.

Soil Pressure Meters (SPM)

31. Soil pressure meters used in this project are Carlson PE-50 stress meters. The stress meters are entirely embedded in concrete, except for the surfaces which monitor the pressure within the soil.

Stainless Steel Plates (SSP)

32. Stainless steel plates consist of 3/8 inch thick 18 inch square steel plates. The plates are grouted to the supporting concrete and secured by means of four 1 foot long rebar studs. Stainless steel plates are used for monitoring of the settlement and deformation of the lock monoliths. In the dewatered condition the elevations of the stainless steel plates are monitored with common leveling techniques. A deep water sounding device is needed for monitoring the elevations when the lock is in operation.

Resteel Strain Gages (RSG)

33. Resteel strain gages measure the axial strain in steel reinforcement bars. The instrumented bars lie near the top and bottom of the concrete slabs. Each set of gages consists of four gages at 90 degree angles around the cross section (see figure 9).

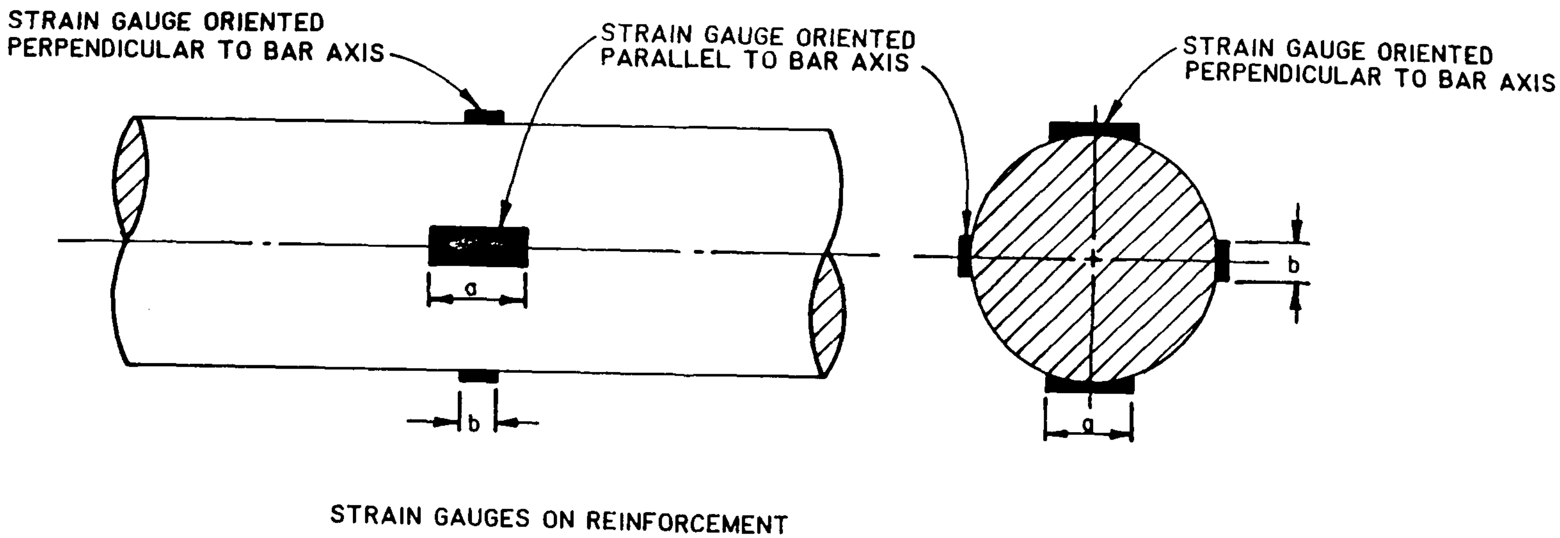


Figure 9. Resteel strain gages (RSG).

Temporary Settlement Points (TSP)

34. For monitoring the settlement during construction of the lock, 28 settlement points were installed in monoliths L-2, L-10, L-16, and L-17. They consist of 1 inch diameter pipe supported by a concrete block and enclosed in a casing of a larger diameter pipe. The casing and the settlement point pipe are both capped for protection. Sections of the pipe were added as construction progressed.

Concrete Stress Meters (CSM)

35. Stress meters are installed in shear keys of the lock to monitor shear key stresses. A shear key connects each monolith of the lock with the adjacent monolith. Concrete stress meters are equipped with internal diaphragms which are deflected by water pressure. A porous stone protects the internal diaphragm from external pressure. Other concrete stress meters are installed at the toe of each wall of the lock and in the middle of the lock slab, since the maximum stress was anticipated at the centerline of the lock. Concrete stress meter are similar in design to the soil pressure meters, but have a higher modulus of elasticity. In the foundation application, concrete stress meters are placed within blocked-out hollows filled with compacted saturated sand.

Settlement Plates (SP)

36. Settlements plates consist of 1 inch galvanized steel pipes supported in soil by 2 feet square steel plates. Each pipe is protected by a casing covered by a cap. Type A settlement plates are located under the concrete structure and type B settlement plates are located in earth fills.

Settlement Points (CSP)

37. The structural design of settlement points is similar to that of settlement plates. Their purpose is to monitor the settlement of the slab of the lock.

Inclinometer (I)

38. Four digital-type inclinometer casings were installed in monoliths L-10 and L-17. The casing bottoms at 10 feet below the base of the lock. Chaining measurements across the lock chambers can be used as crosschecks of the inclinometer readings. After the inclinometers were installed, two additional inclinometers were added because of slope failures which occurred in July 1979 and April 1980.

Surveying Monuments

39. For the construction of Lock and Dam No. 1 four bench marks were used:

U 302 (stamped U302 1977)
  Latitude 311512 N
  Longitude 0915934W
  Located 15.5 miles NE from city of Effie, Catahoula Parish, LA
  Consisting of: Copper clad steel rod

PBM RR1 USE (stamped PBM RR LN1)
  Latitude 311456N
  Longitude 0915907W
  Located 15.9 miles NE from city of Effie, Catahoula Parish, LA
  Consisting of: Galvanized steel pipe

PBM RR1 B USE (stamped PBM 1B)
  Latitude 311443N
  Longitude 0915825W
  Located 16.6 miles NE from city of Effie, Catahoula Parish, LA
  Consisting of: Galvanized steel pipe

PBM RR1 USE (stamped PBM 1C)
  Latitude 311516N
  Longitude 0915821W
  Located 17.2 miles NE from the city of Effie, Catahoula Parish, LA
  Consisting of: Galvanized steel pipe

40. There are six bench marks in the immediate vicinity of Lock and Dam No. 1. The bench mark PBM 1-27-32 is located in the parking area of the project office near the entrance from LA highway 3102. This bench mark is used for survey of the surface monuments (SM) and reference bolts (RB). An unidentified bench mark is located within the fenced-off area of former Corps of Engineers offices about 0.4 miles from the present office. On the east bank of the Red River are two bench marks. At the northern end of the levee road there is PBM 1-27-33. Approximately 1800 ft north of this bench mark is PBM-1. These bench marks are used for surveying the closure dam. PBM RR-1-C is located on an unsurfaced road to Lake Larto about 0.8 miles from Lock and Dam No. 1 off LA Highway 3102. PBM-RR-LDM-1 is located 250 ft south of LA Highway 3102 in a housing area where the road to Larto Lake branches off.

Calibration of Gages

Strain gages and resteel strain gages

41. The strain gages and resteel strain gages are provided with a gage factor by the manufacturer and therefore no calibration is necessary in the field. The active gage is read directly from connecting wires on a scale of micro inches. Active gages are placed in groups of four forming a cross and connected by wires in the form of a Wheatstone bridge (see figure 9). The gages placed in the direction of the longitudinal axis provide direct reading of strain amplified by a factor of 2. Gages placed in the perpendicular direction render data on Poisson's Ratio which must be reduced by a factor of 2.5.

Soil pressure meters

42. Carlson PE-50 soil stress meters have a gage range of 50 psi. They measure stress to the nearest 0.2 psi. All stress meters were calibrated by the manufacturer and recalibrated by WES. The calibration by the manufacturer was done using a rigid plate applied uniformly to the membrane. The WES calibration was done by the application of a uniform pressure through a flexible diaphragm. To check for water leaks and drift, further calibration was provided by WES by submerging the meters in a water tank subjected to 40 psi for a duration of 16 to 17 hours. After that, the meter and whole cable were submerged in water under 40 psi hydrostatic pressure for 24 hours. Next, the SPM

was pressured hydrostatically in 10 psi increments to a maximum pressure of 40 psi. Load calibration of the diaphragm was done pneumatically in two cycles. In the first cycle pressures were increased in 10-psi increments to 40 psi and then decreased in 10 psi decrements to 0. In the second cycle, the pressure was increased to a maximum pressure of 40 psi and then decreased to 0 psi. Calibration constants were computed from the average slope of the curve for increasing values.

Concrete stress meters

43. Concrete stress meters have a range of 0 to 800 psi in compression. The concrete stress meters were calibrated by the manufacturer and tested at WES for leakage and low resistance. The calibration constants were received from the manufacturer but were not verified by WES. The maximum loading condition available at WES is 100 psi. Since the meter stress range is beyond the range of WES testing equipment, the calibration constants were not changed.

Monitoring of the Water Condition

44. Water surface elevation is monitored by means of pool gages. The pool gages are read on a daily basis. The upper pool gage is located in the upstream end of the upper guardwall. The tail water gage is located on the downstream end of the lower guardwall. The pool gage water surface elevation station is located several miles upstream of the structure. Figure 10 gives the hydrostatic data from September 1984 until December 1986.

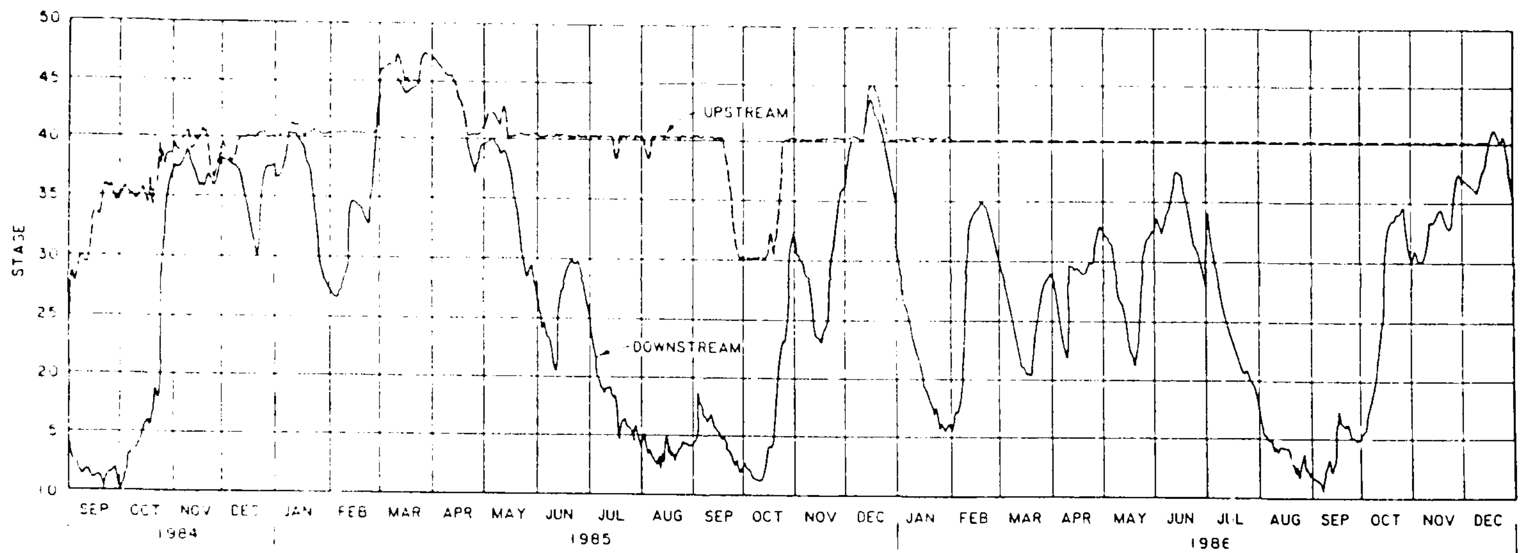


Figure 10. Hydrograph data from September 1984 to December 1986.

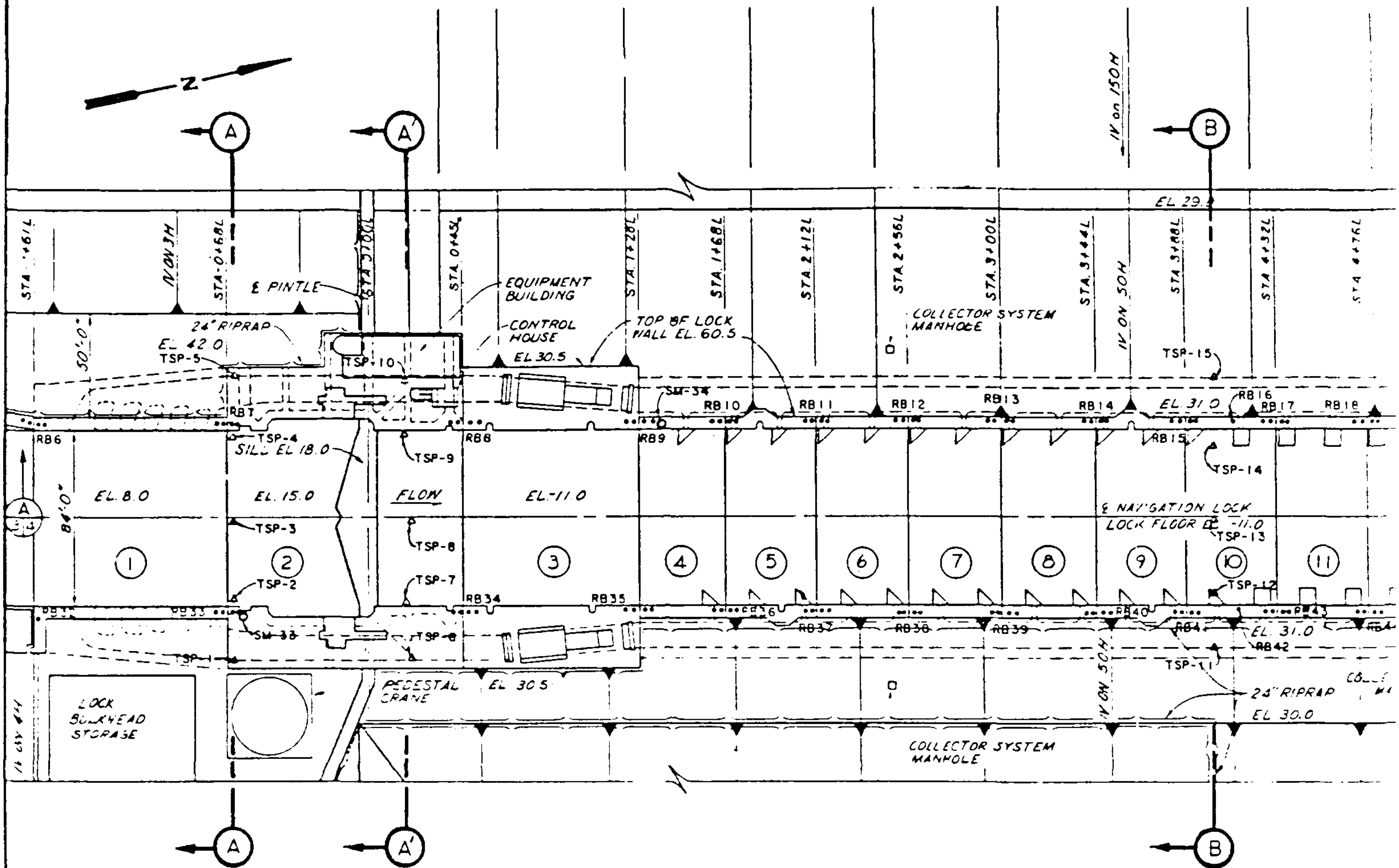
PART V: SETTLEMENT

Settlement

Observed and predicted settlement

45. Plan view for the special instruments used in settlement analyses are shown on plate 4 and in figure 11. Base slab settlement, predicted settlement for Cases I and II, construction and operation conditions, along with the observed foundation settlements at monolith L-2, L-10, L-16, and L-17, are plotted in figures 12 through 17. Figure 18 shows settlements along the center of the lock. The Case I condition assumed the lock structure to be complete with backfill that was brought up concurrently with the walls. Actual placement of backfill sometimes lagged nearly six months after the concrete was placed. Completion dates for each lift at these monoliths are shown in figures 19 through 21. Readings of all instrumentation were to be made when conditions were similar to those assumed in design. The assumed condition for Case I actually occurred in May 1983. Case II condition assumed the lock in operation with upper pool elevation at 40.0 feet and lower pool elevation at 4.0 feet. Although the stainless steel plates were not read until September 1985, after approximately a year in service, when the lock was dewatered, these data was considered acceptable for Case II.

46. With only the base slab in place, the settlement profile across the width of the lock followed the expected dish-shape pattern, with more settlement occurring at the center than at the sides. However the plots did not reverse in shape to reflect the placement of the walls and backfill. The settlement profile should have shown more settlement at the sides than at the center of the lock. The observed data were not in agreement with the expected settlement. The inconsistency may be due to unreported settlement readings prior to the addition of riser pipes during the installation period from April 1981 through March 1982. The maximum settlement, as recorded in field data, was 2.25 inches in March 1981; an additional settlement of 1.90 inches was observed for the lock by September 1985. Ultimate settlement predicted was 4.50 inches; total settlement, as observed in September 1986, was 4.15 inches.



A
A'
B
N
EL 29.5
STA 0+61L
STA 0+06L
STA 0+45L
STA 1+28L
STA 1+68L
STA 2+12L
STA 2+56L
STA 3+00L
STA 3+44L
STA 3+88L
STA 4+32L
1V ON 150H
1V ON 50H
℄ PINTLE
EQUIPMENT BUILDING
CONTROL HOUSE
TOP OF LOCK WALL EL. 60.5
COLLECTOR SYSTEM MANHOLE
24" RIPRAP
EL 42.0
EL 30.5
EL 31.0
SILL EL 18.0
EL. 8.0
EL. 15.0
FLOW
EL -11.0
84'-0"
50'-0"
℄ NAVIGATION LOCK
LOCK FLOOR EL -11.0
LOCK BULKHEAD STORAGE
PEDESTAL CRANE
24" RIPRAP
EL 30.0
COLLECTOR SYSTEM MANHOLE
TSP-1
TSP-2
TSP-3
TSP-4
TSP-5
TSP-6
TSP-7
TSP-8
TSP-9
TSP-10
TSP-11
TSP-12
TSP-13
TSP-14
TSP-15
SM-33
SM-34
RB6
RB7
RB8
RB9
RB10
RB11
RB12
RB13
RB14
RB15
RB16
RB17
RB18
RB33
RB34
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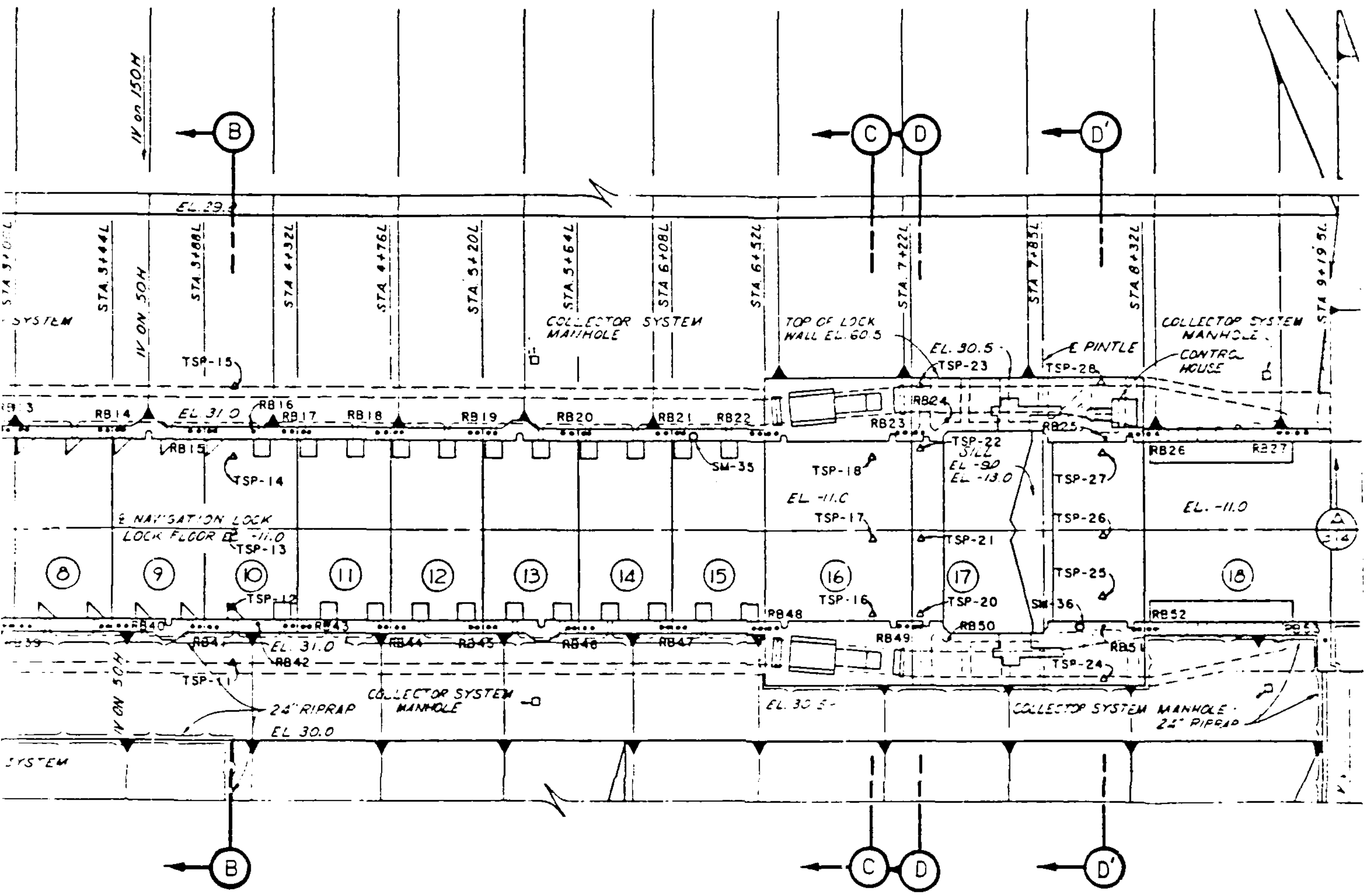


RED RIVER WATERWAY, LA., TEX., ARK., OKLA
MISSISSIPPI RIVER TO SHREVEPORT, LA.

PLAN OF INSTRUMENTS IN LOCK

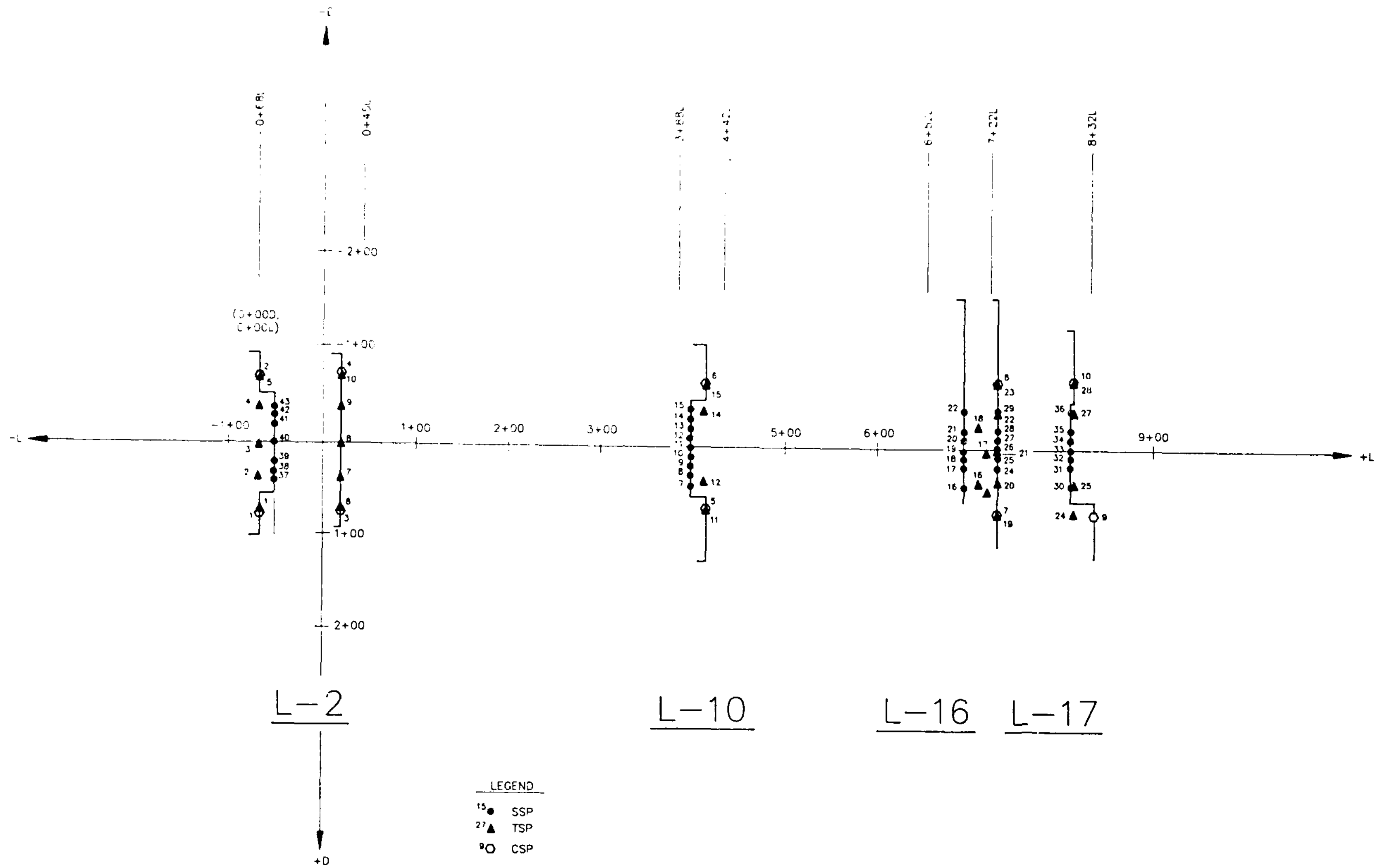


Figure 11. Plan view of special instrumentation in the lock.

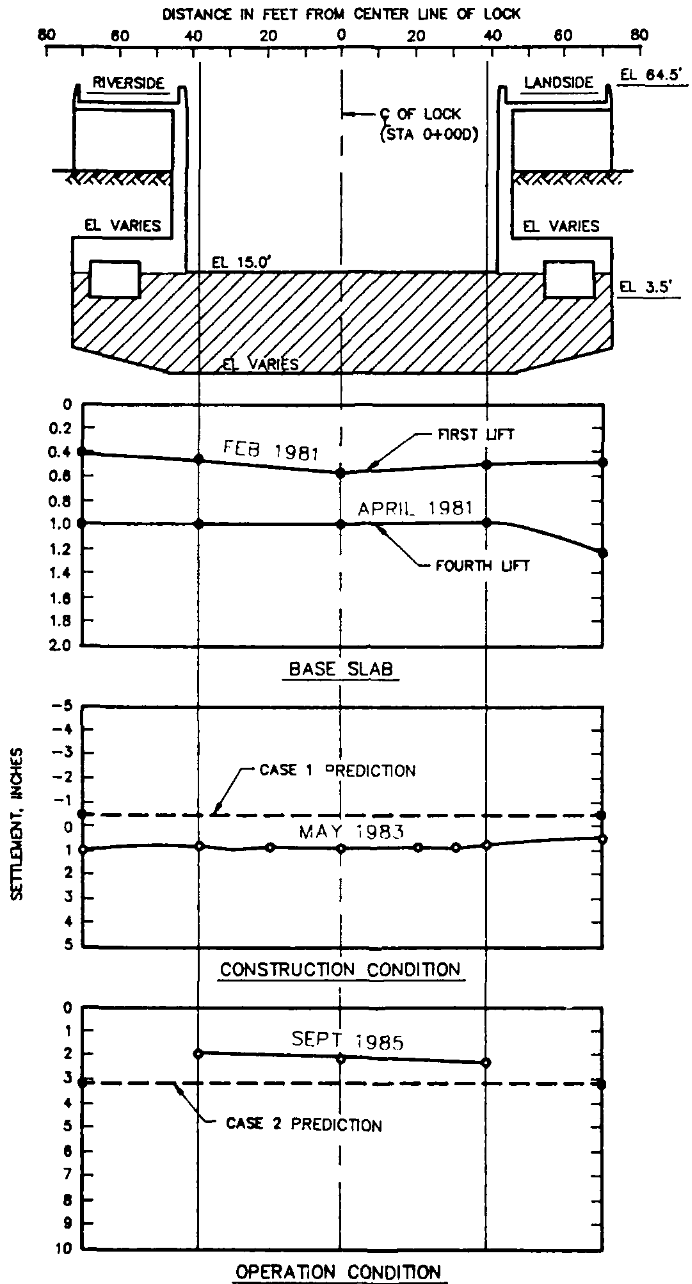


Figure 12. Settlement at lock monolith L-2 station -1+65L.

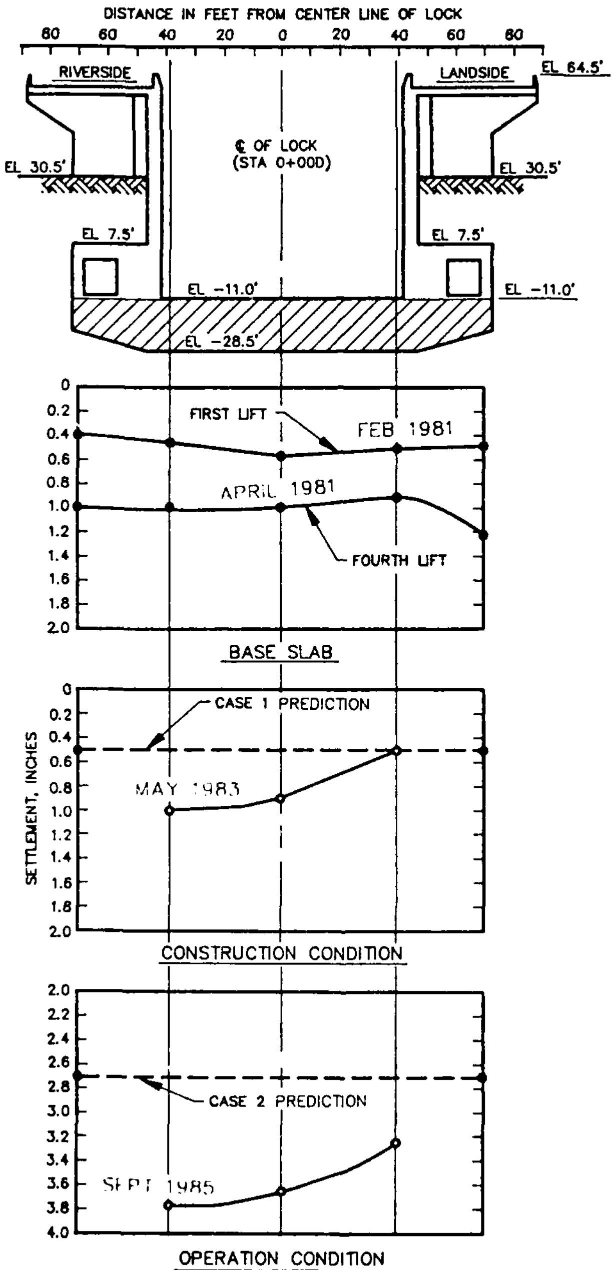


Figure 13. Settlement at lock monolith L-2 station 0+20L.

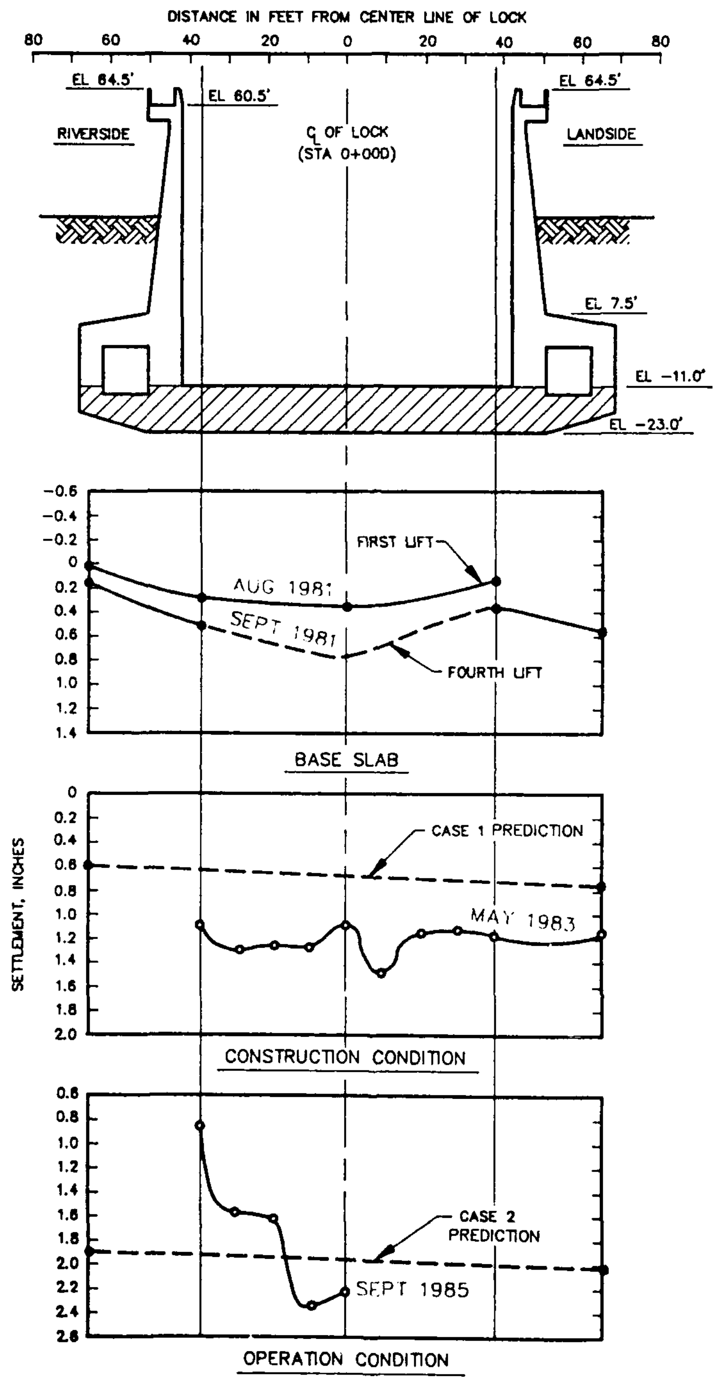


Figure 14. Settlement at lock monolith L-10 station 4+10L.

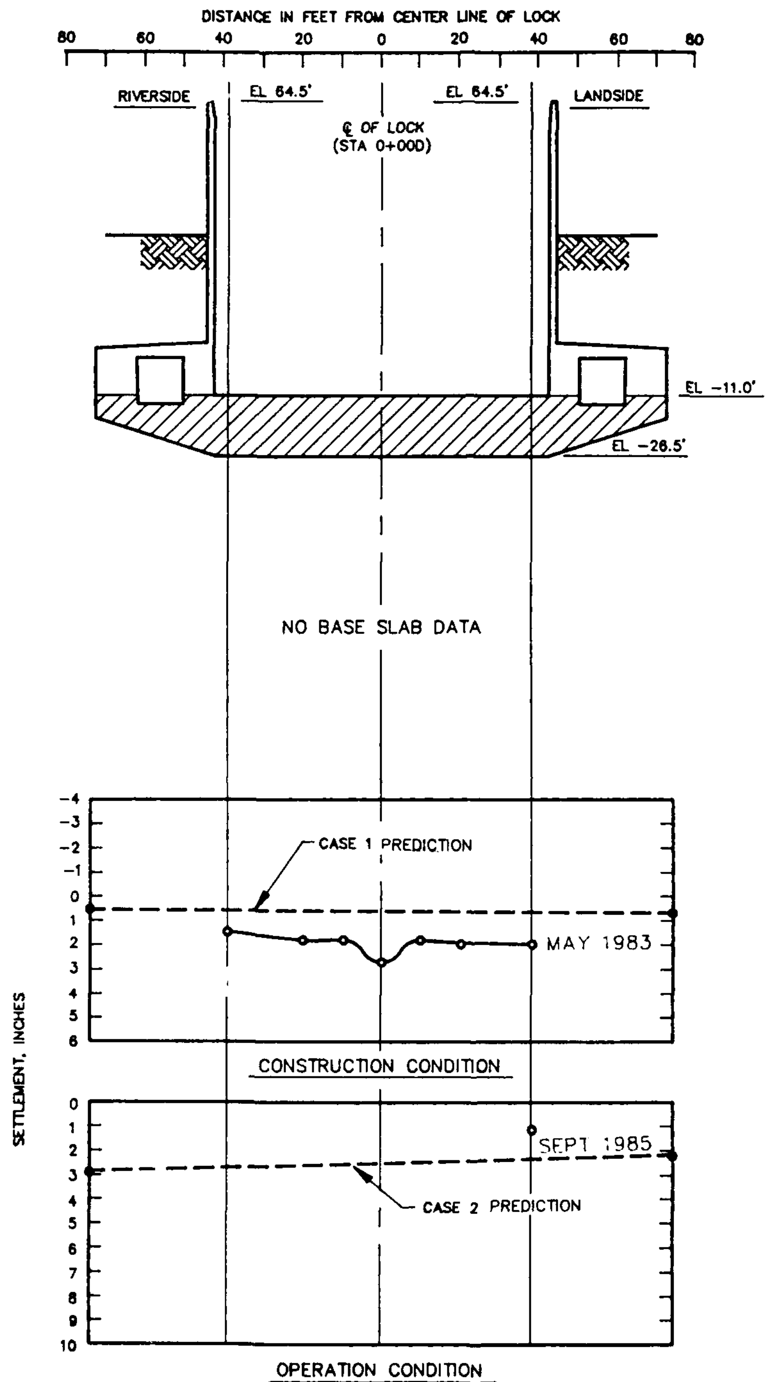


Figure 15. Settlement at lock monolith L-16 station 7+12L.

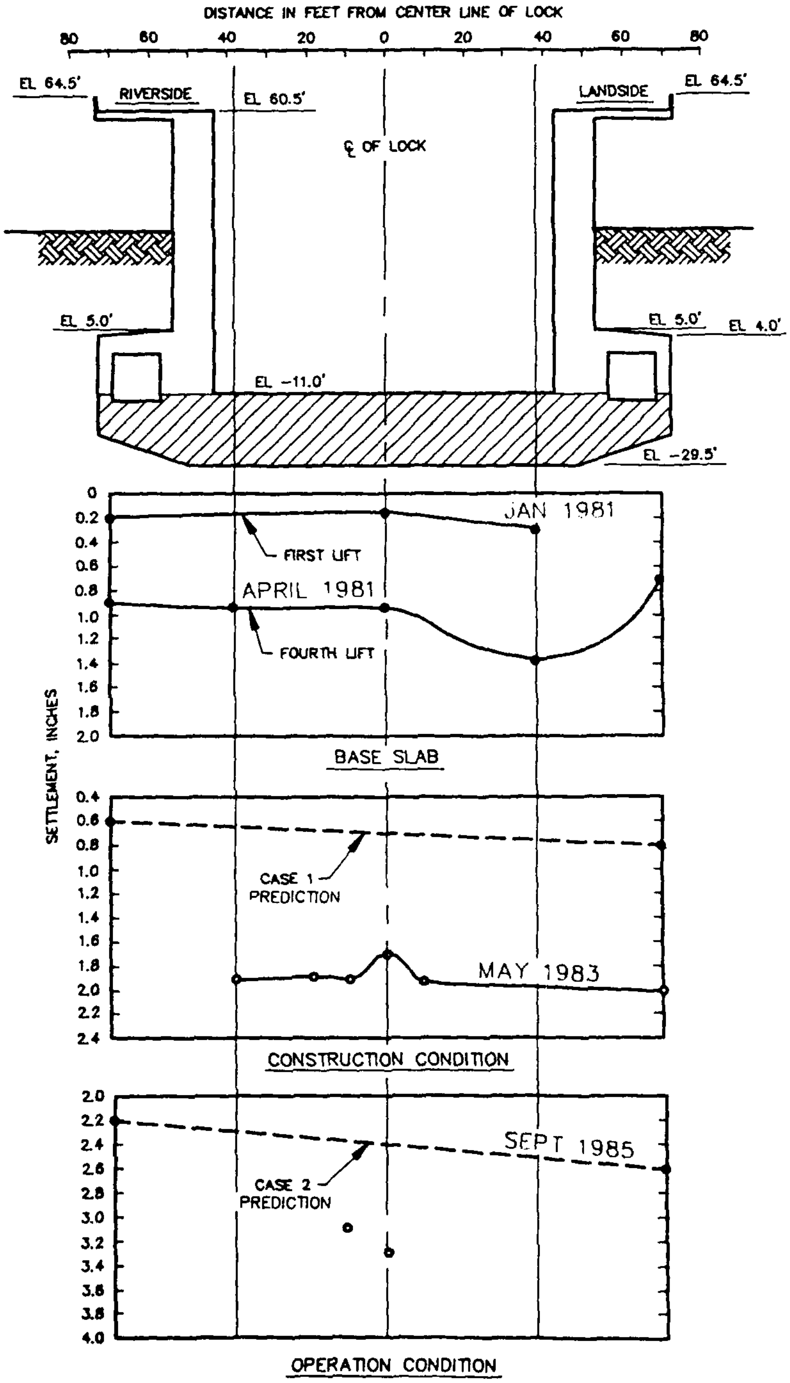


Figure 16. Settlement at lock monolith L-17 station 7+34L.

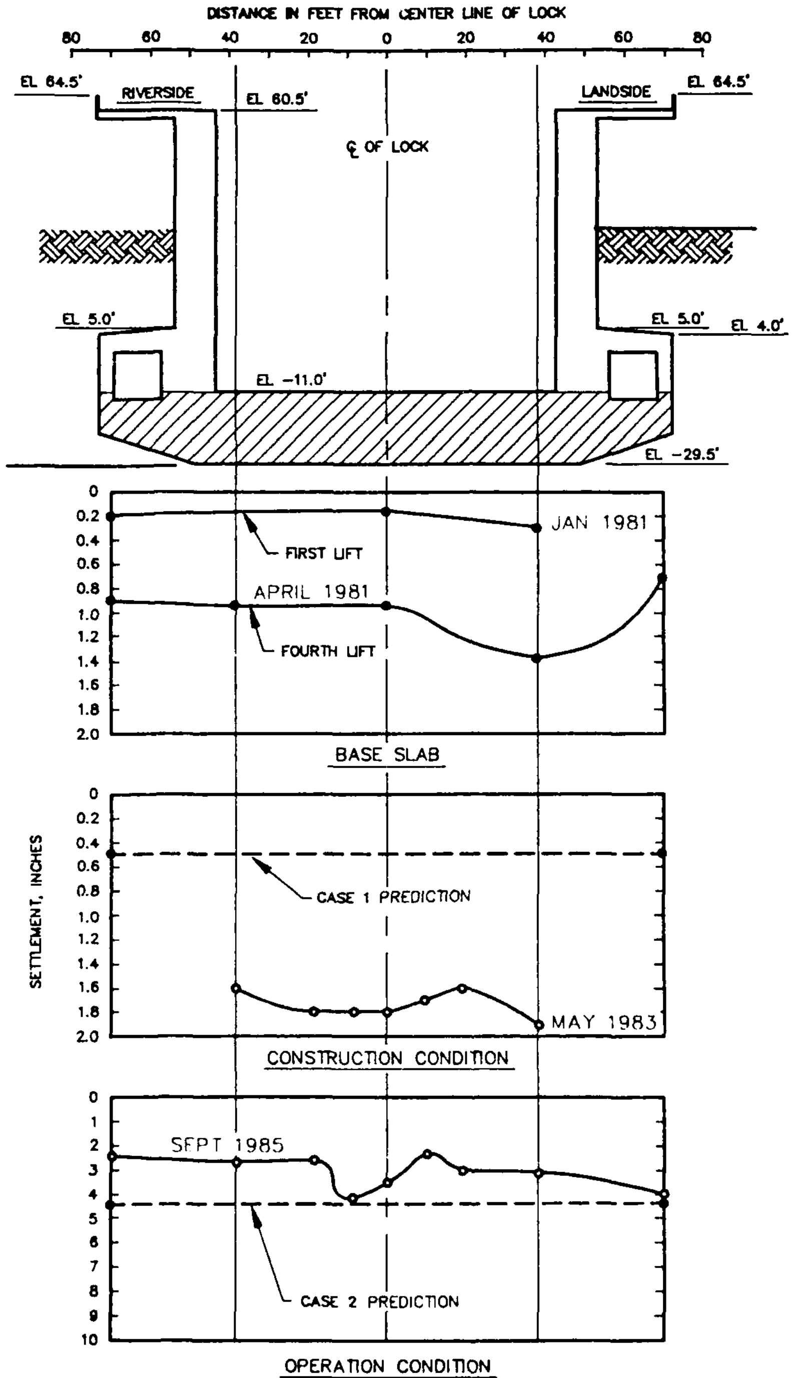


Figure 17. Settlement at lock monolith L-17 station 8+12L.

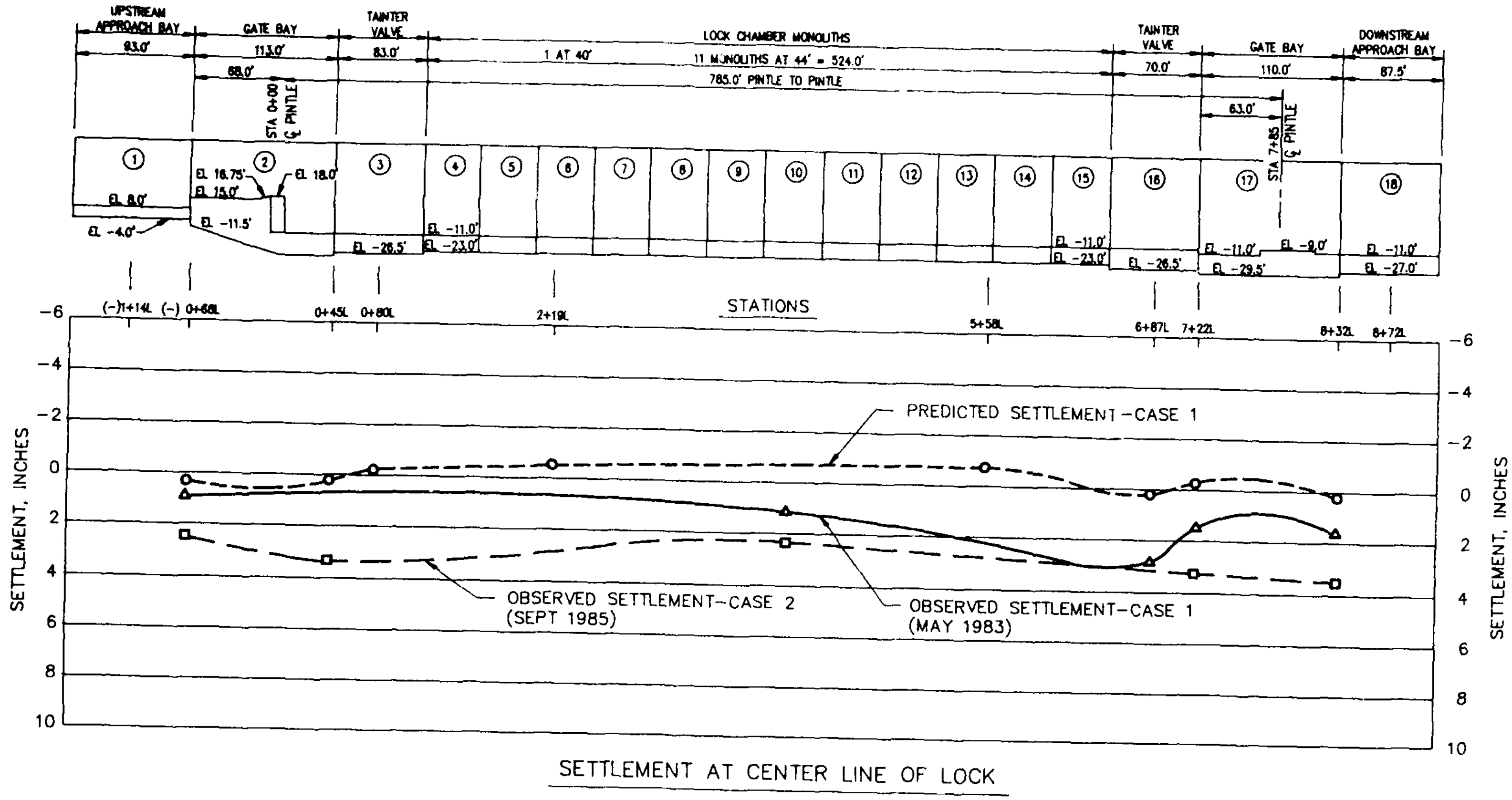


Figure 18. Settlement at center line of the lock. (Initial reading May 1982)

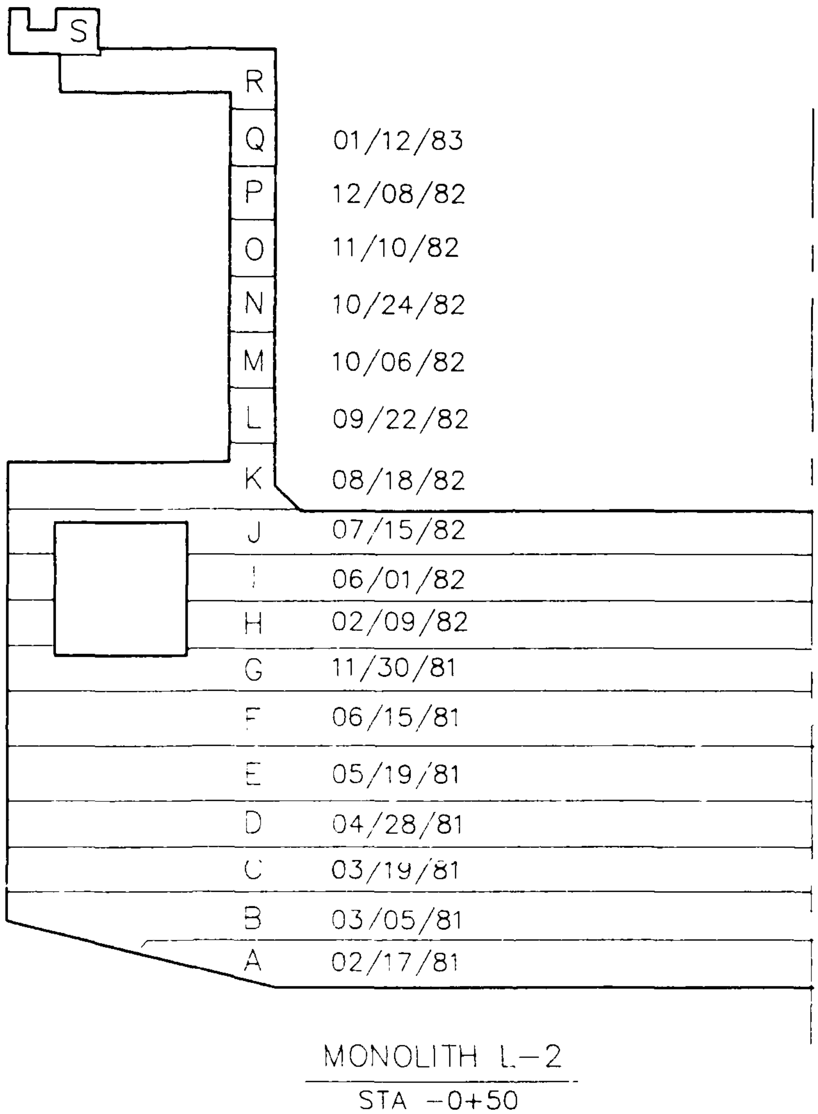


Figure 19. Dates of completion for each lift.

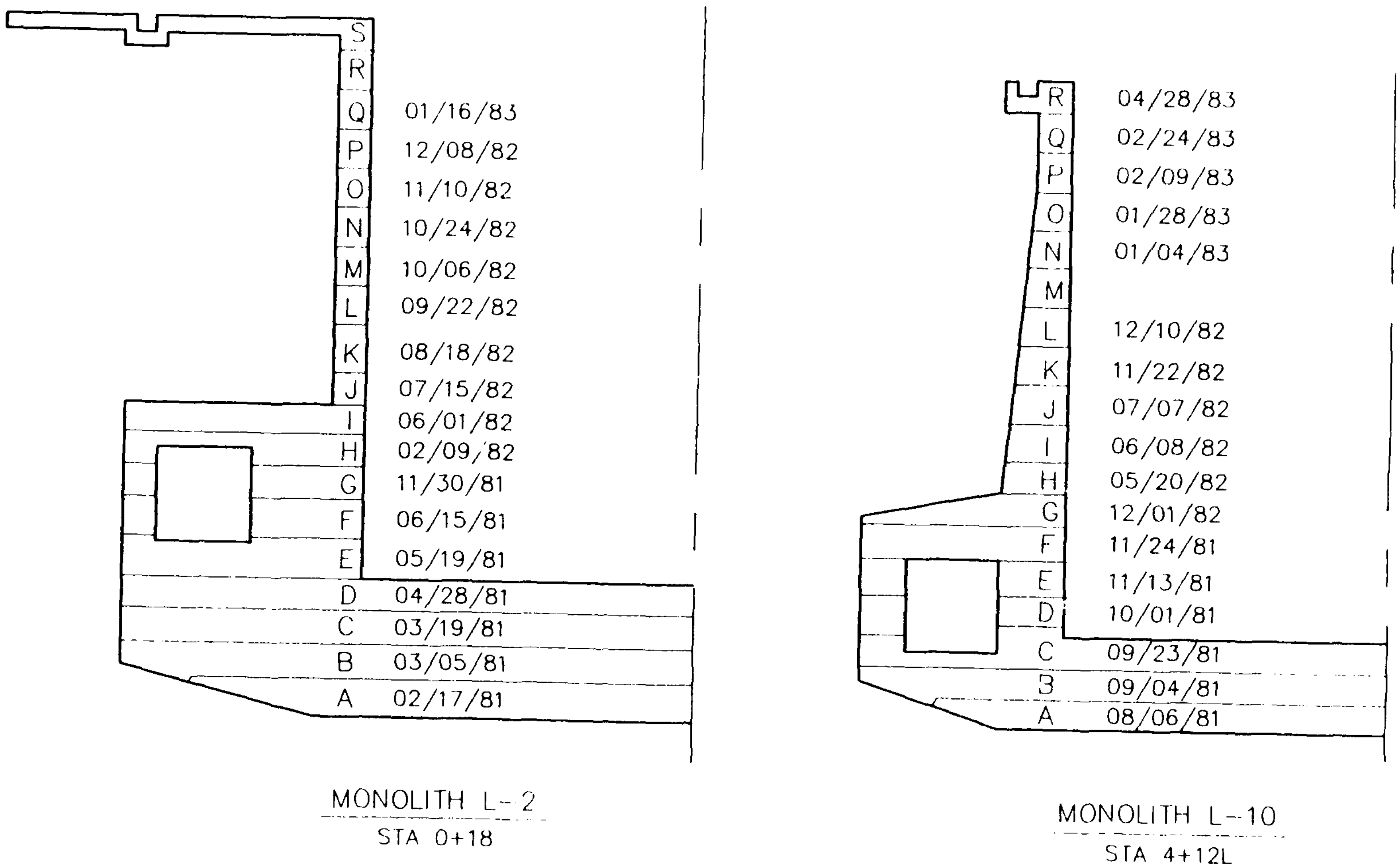


Figure 20. Dates of completion for each litt.

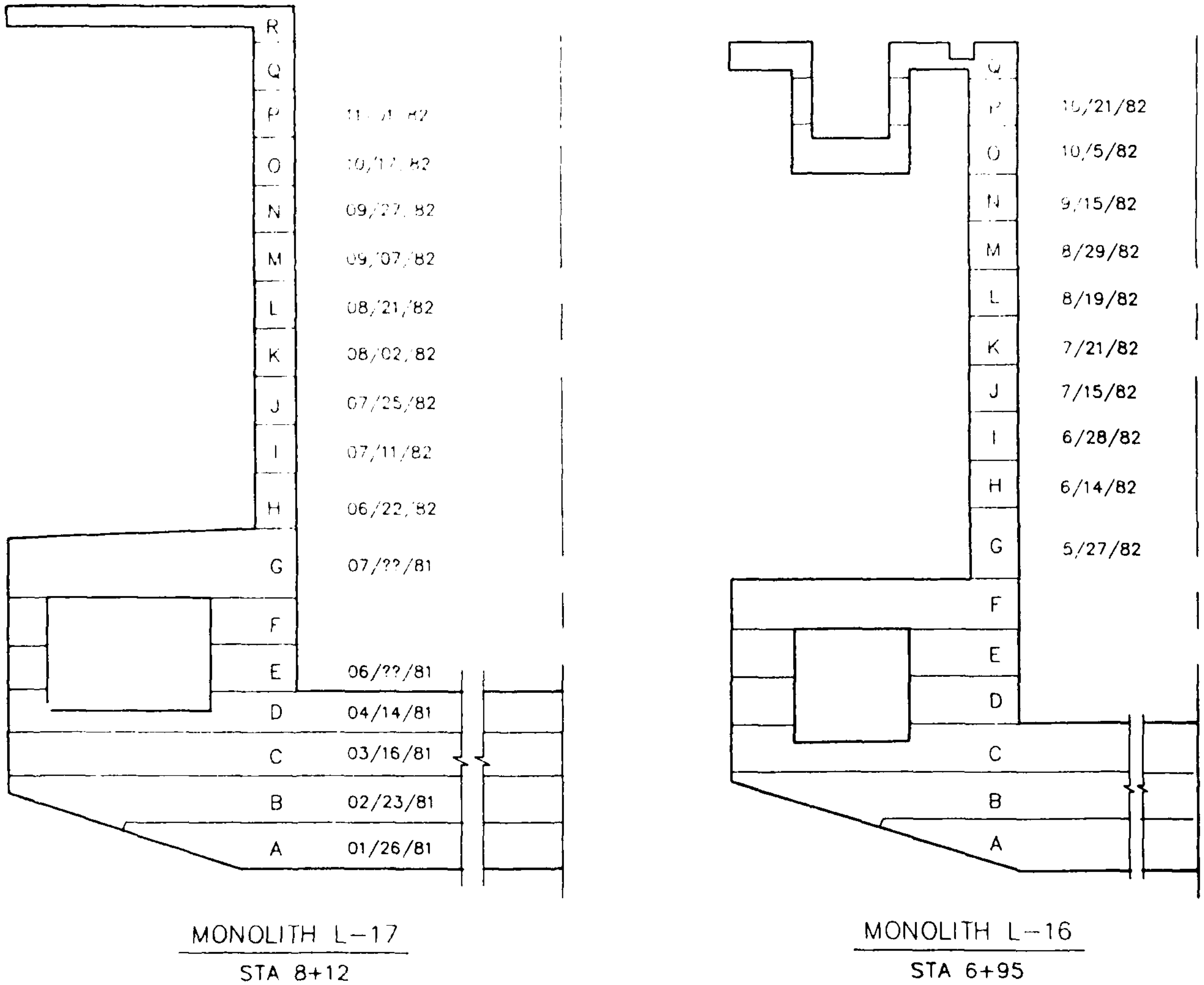


Figure 21. Dates of completion for each lift.

Settlement and movement at wall joints

47. The settlement of the top of the monolith walls is shown in figure 22. The settlement at the top of the walls was relatively uniform, with maximum settlement occurring at the gate bay monoliths. Figure 22 reflects differential settlement between the monoliths; this is believed to be a survey error since all the lock monoliths were structurally keyed together. This may also be explained by the differences in the individual monolith completion time. Figure 23 shows the transversal movements at the wall joints when the lock was dewatered and operating. Very little movement at the wall joints has occurred. The reference bolt readings for the horizontal movements indicated that some joints were opening while others were closing. The maximum observed joint opening was 0.29 inches and the maximum amount of closing was 0.03 inches. Figures 24 and 25 show some detail of the joints at the floodwall and a lock monolith. There have been no changes in these movements since last observed in January 1987.

Settlement at the closure dam

48. The surface monuments at the closure dam were installed 3 feet above the ground. The monuments are set in concrete surrounded by four 6 inch square wooden posts. Monuments SM-1 through SM-4 are spaced approximately 100 feet apart. Monuments SM-5 and SM-6 are located on the toe of the levee (see figure 26). Measurements for SM-8 and SM-9 were not reported. Maximum settlement at the closure dam was 1.37 feet (see figure 27).

Settlement at the dam abutment piers

49. Figure 28 shows the total settlement along the dam piers. The settlements were calculated from settlement plate data from April 1981 through September 1986. Some readings were questionable because of unreported elevations prior to the extension of pipes. The settlement plot taken from surface monument data was not reported until October 1984. The upper plot in figure 29 shows settlements from October 1984 through September 1986. The lower plot in figure 29 shows a comparison of settlements between the surface monument and the settlement plate from 17 April 1984 to 19 September 1986. The instrumentation readings were in close agreement.

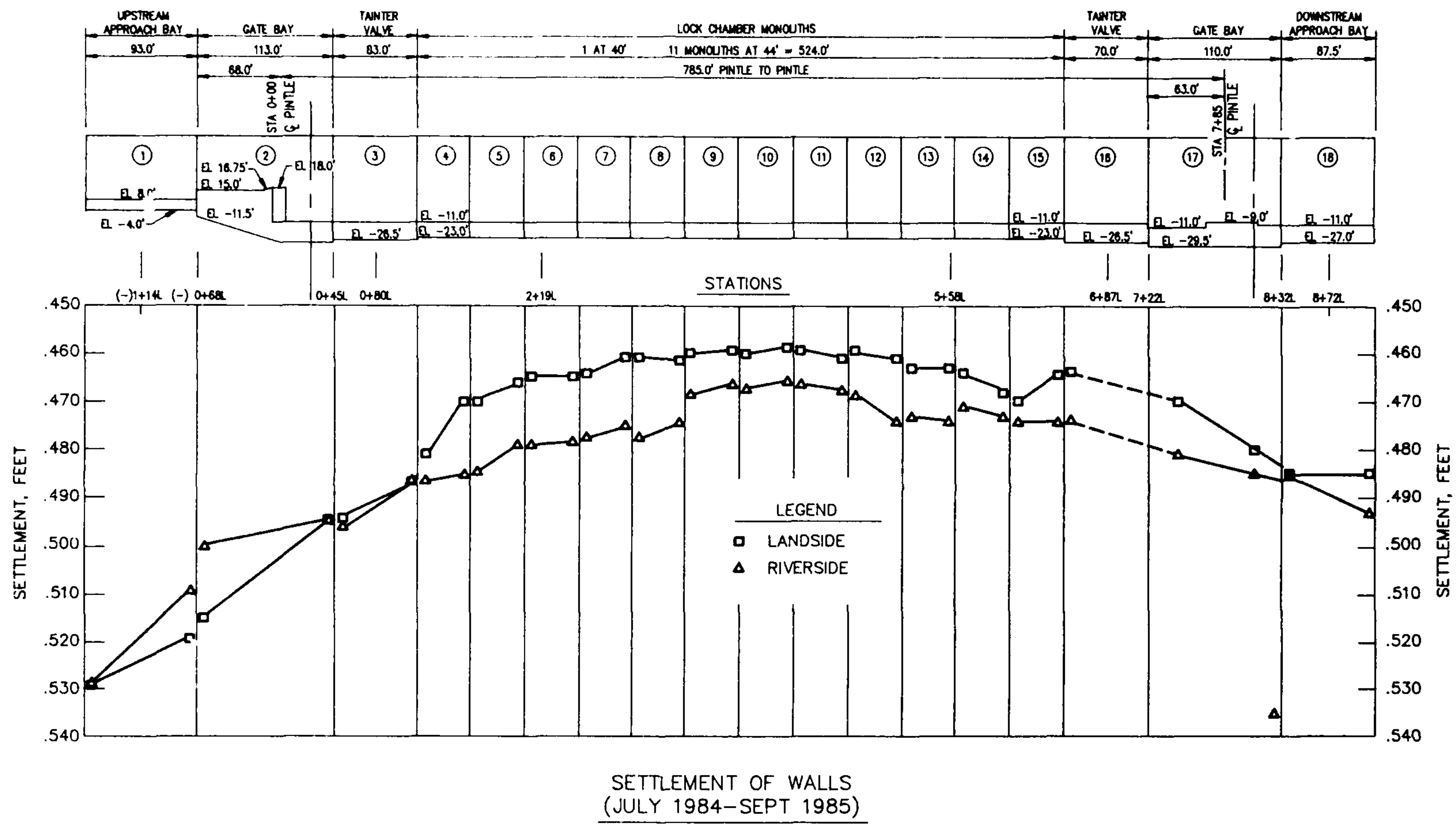


Figure 22. Settlement at top of the lock walls.

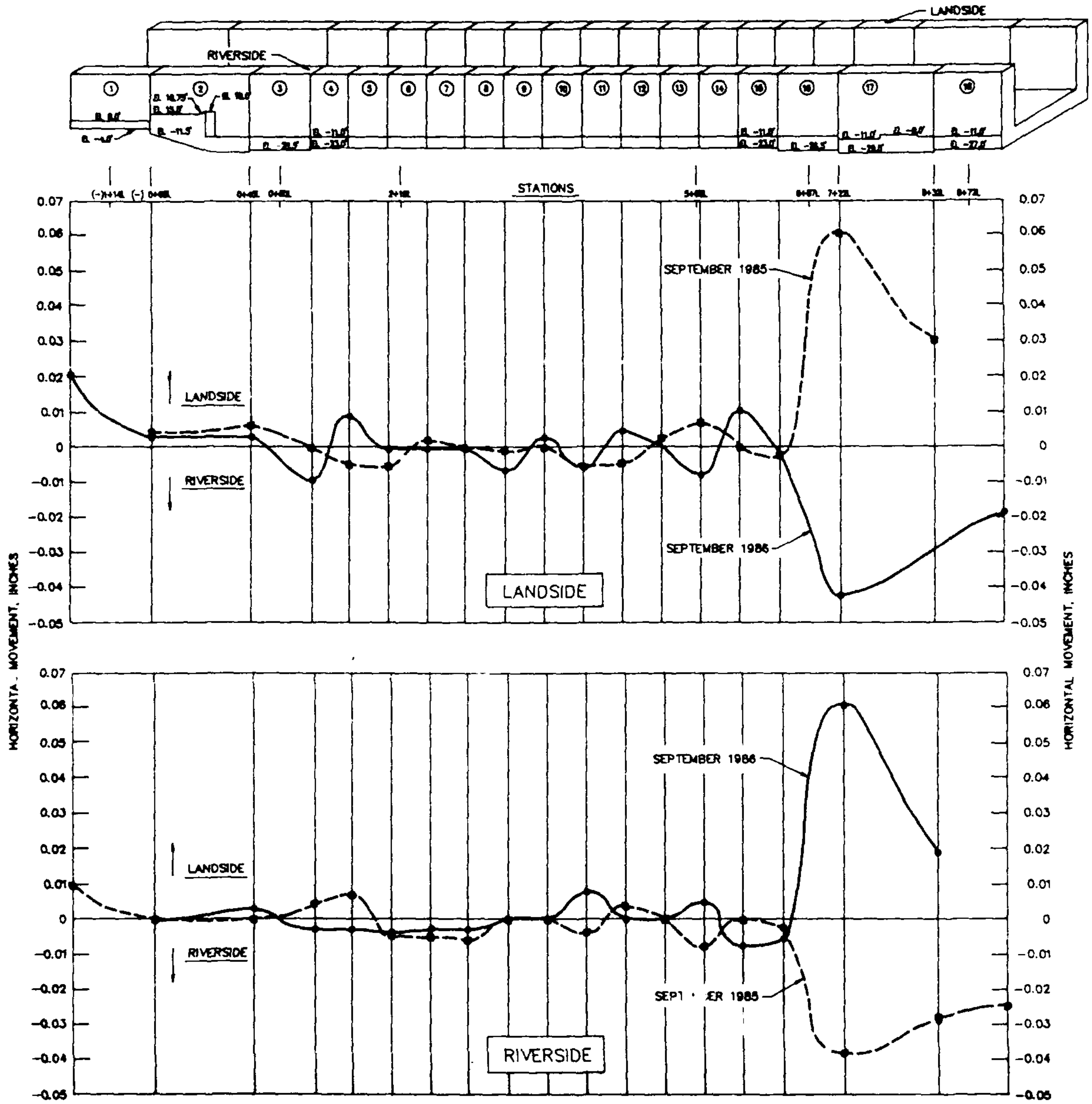


Figure 23. Transversal movement at wall joints during dewatered and operating conditions.

Figure 24. Observed joint opening in the floodwall.

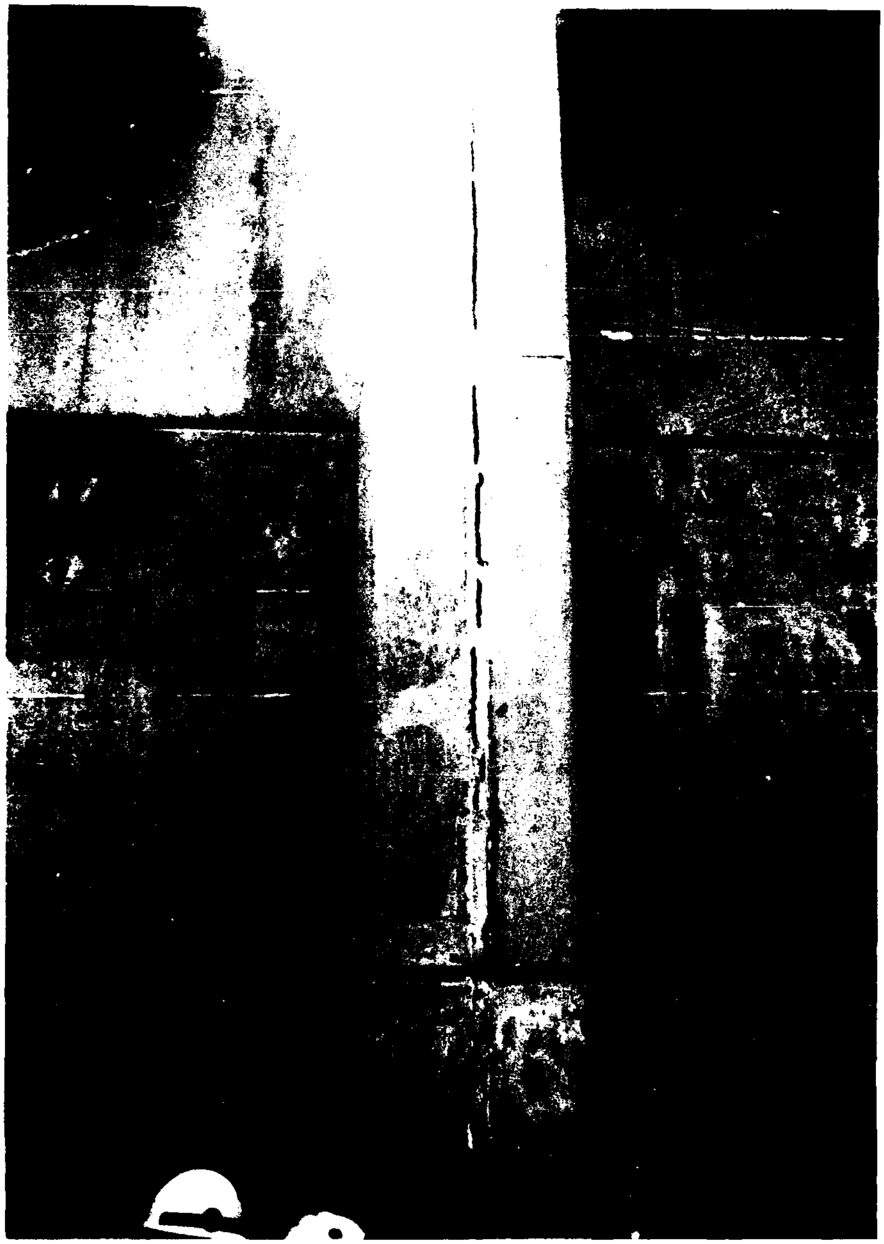


Figure 25. Observed joint opening in the lock wall.

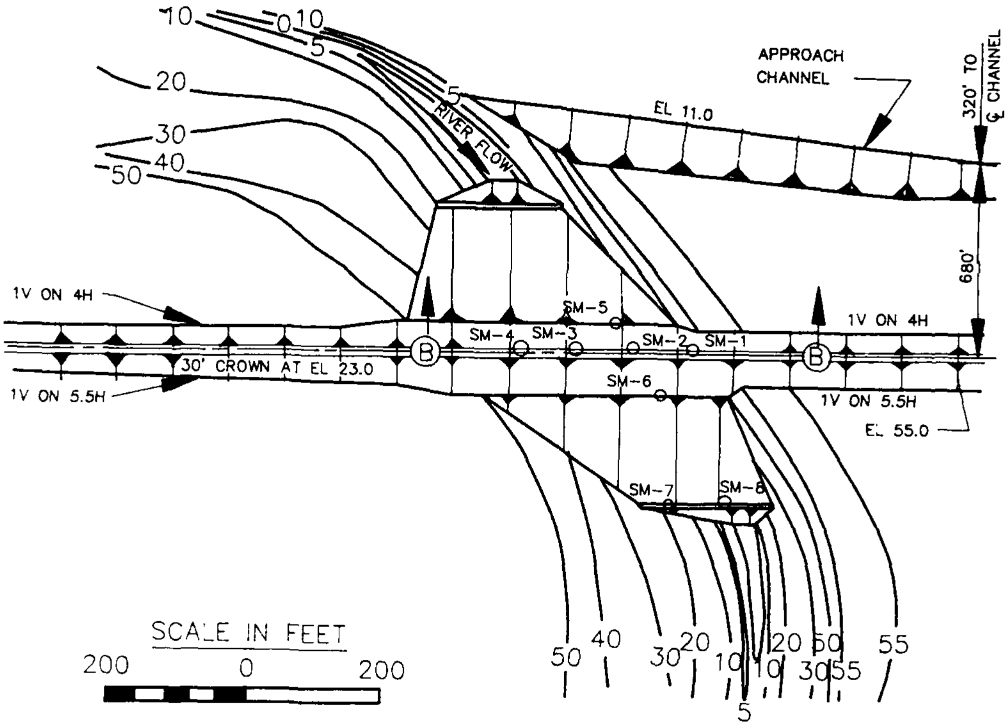


Figure 26. Plan view of instrumentation in the closure dam.

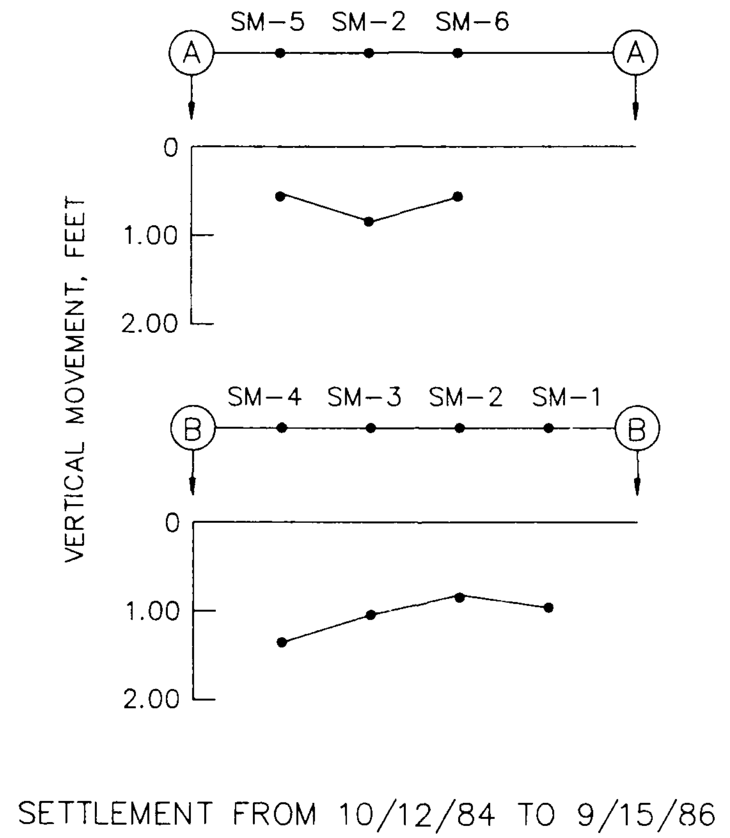


Figure 27. Closure dam settlement.

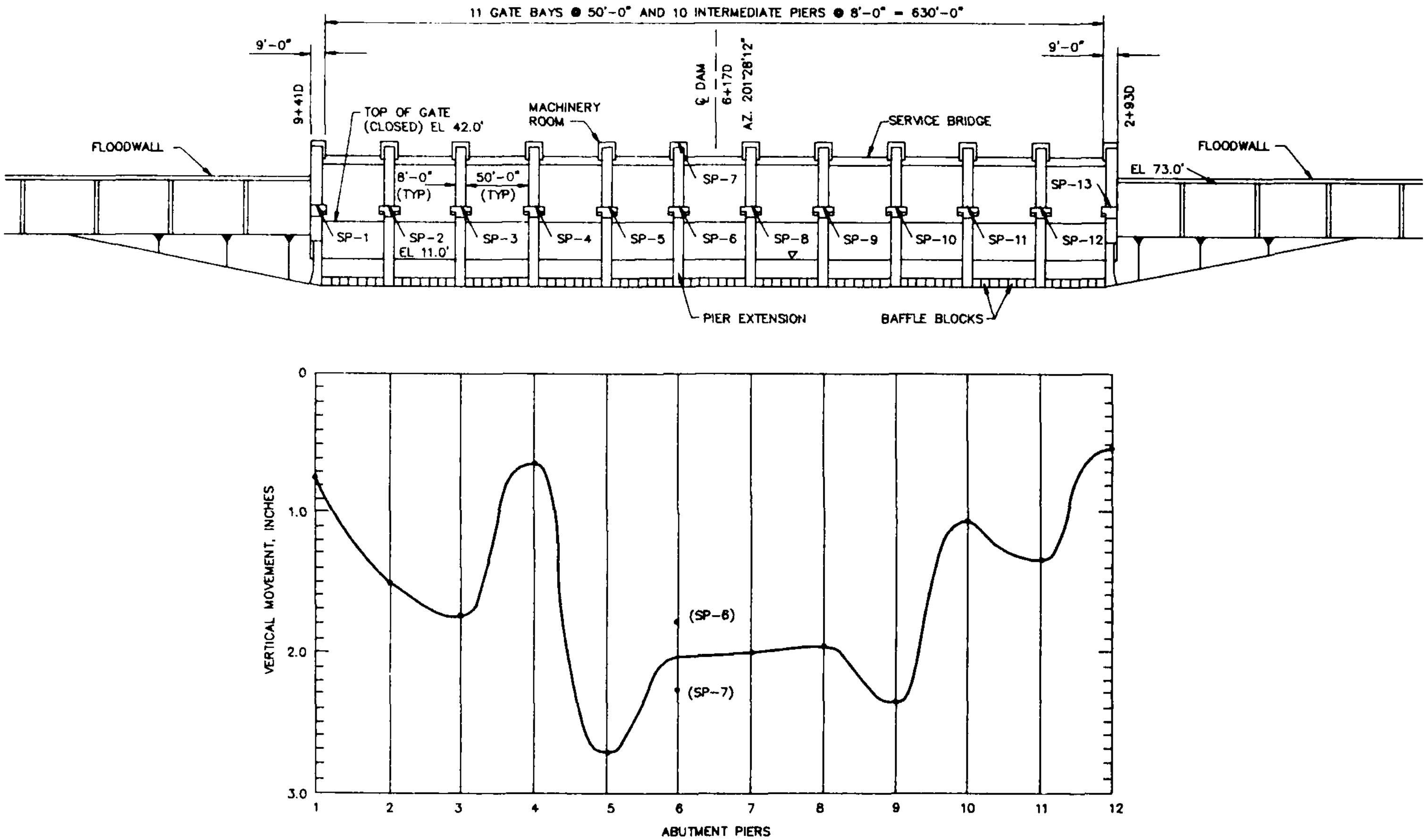


Figure 28. Settlement plate data for dam piers.

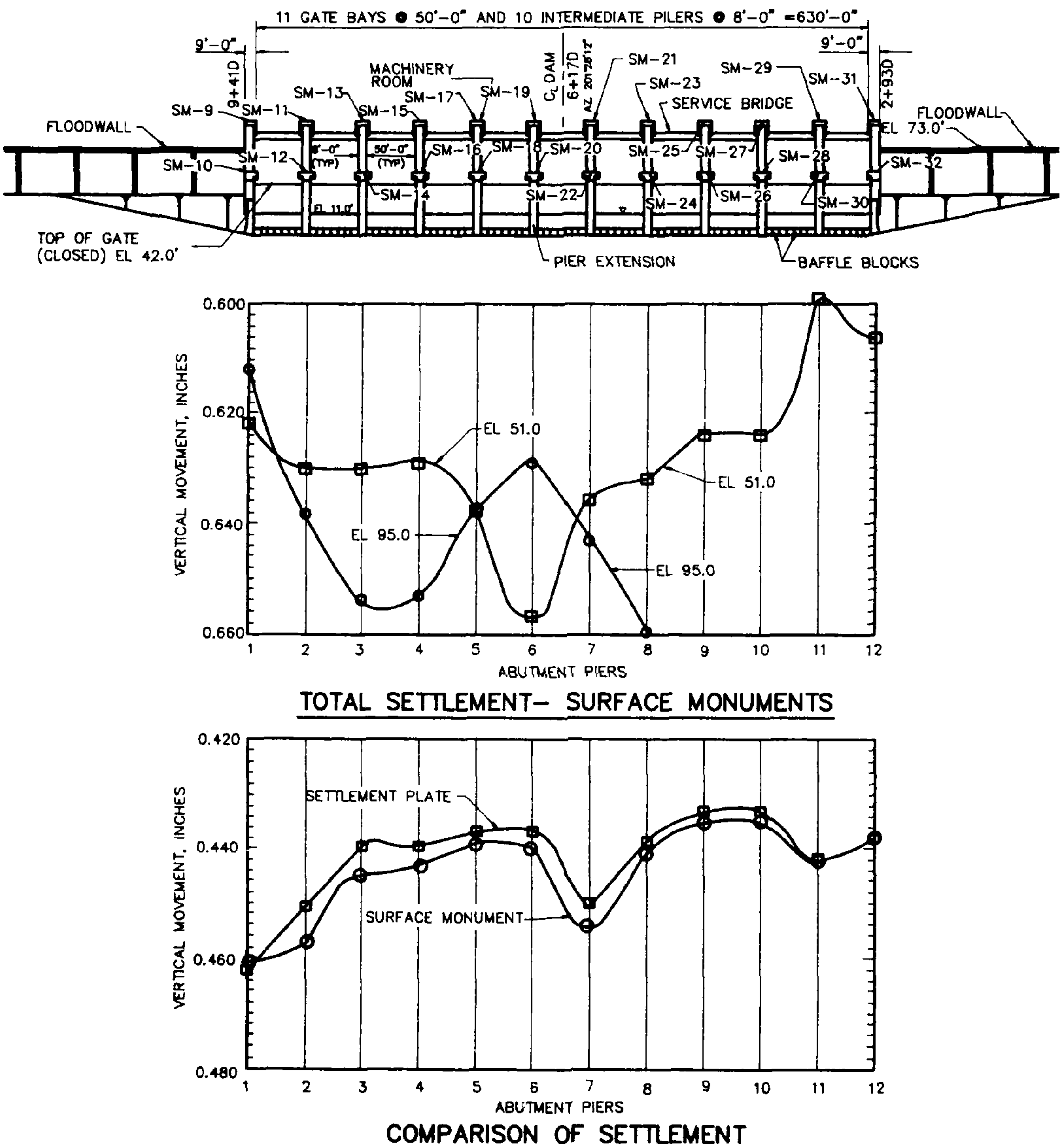


Figure 29. Surface monument data for settlements along dam piers.

PART VI: EARTH PRESSURES AND WALL MOVEMENTS

Backfill

50. The backfill and drainage materials placed adjacent to the lock and dam consisted primarily of select sand, select clay, and random backfill. The select sand backfill was placed along both sides of the lock structure. Placement of the select sand started in December 1981 and was completed in July 1983. A neoprene coated nylon fabric was placed on the natural ground under the select sand. Figure 30 shows a typical backfill section. The in-place density was determined using the standard sand cone per ASTM D-1556. The material was spread and compacted with the track of a crawler-type tractor and the material against the structure was compacted using a hand operated vibratory compactor. Refer to plate 2 for design parameters.

51. Two sources of sand were mixed to meet the gradation requirements. The materials were obtained from the Shannon dredging site and pit run sand.

52. The select clay was designed to provide a 5 foot thick impervious layer over the select sand. Eight selected clay cutoff walls were constructed perpendicular to the lock axis which divided the clay blankets into six impervious cells surrounding the sand. Placement of the select clay cutoffs against the structure was not started until nearly 6 months after the concrete was placed.

53. The random backfill consisted primarily of silty clay, clayey silt, and clay. Little or no silty sand was used.

Observed Earth Pressures

Earth pressures versus time

54. The time plots of the earth pressures between the backfill and the lock wall, as measured by soil pressure meters were presented in the "Data Summary" in figures B-61 to B-64 and B-74 to B-79. All functional soil pressure meters generally exhibit the same responses. The pattern could be associated to specific events as shown in figure 31 (see figure 35 for location of the soil pressure meters). At the beginning of construction the instruments indicate an increase of soil pressure corresponding to the weight of the lock

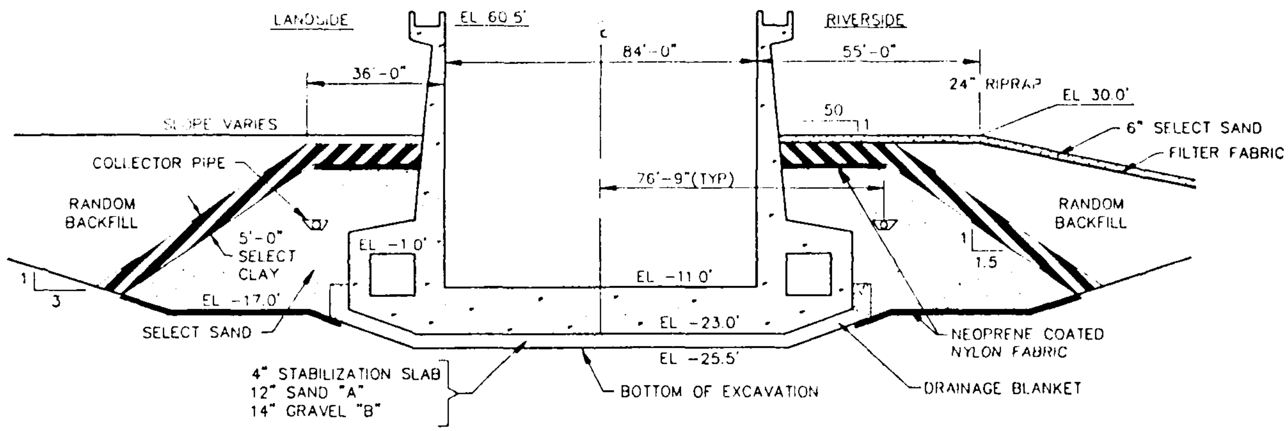


Figure 30. Typical backfill section.

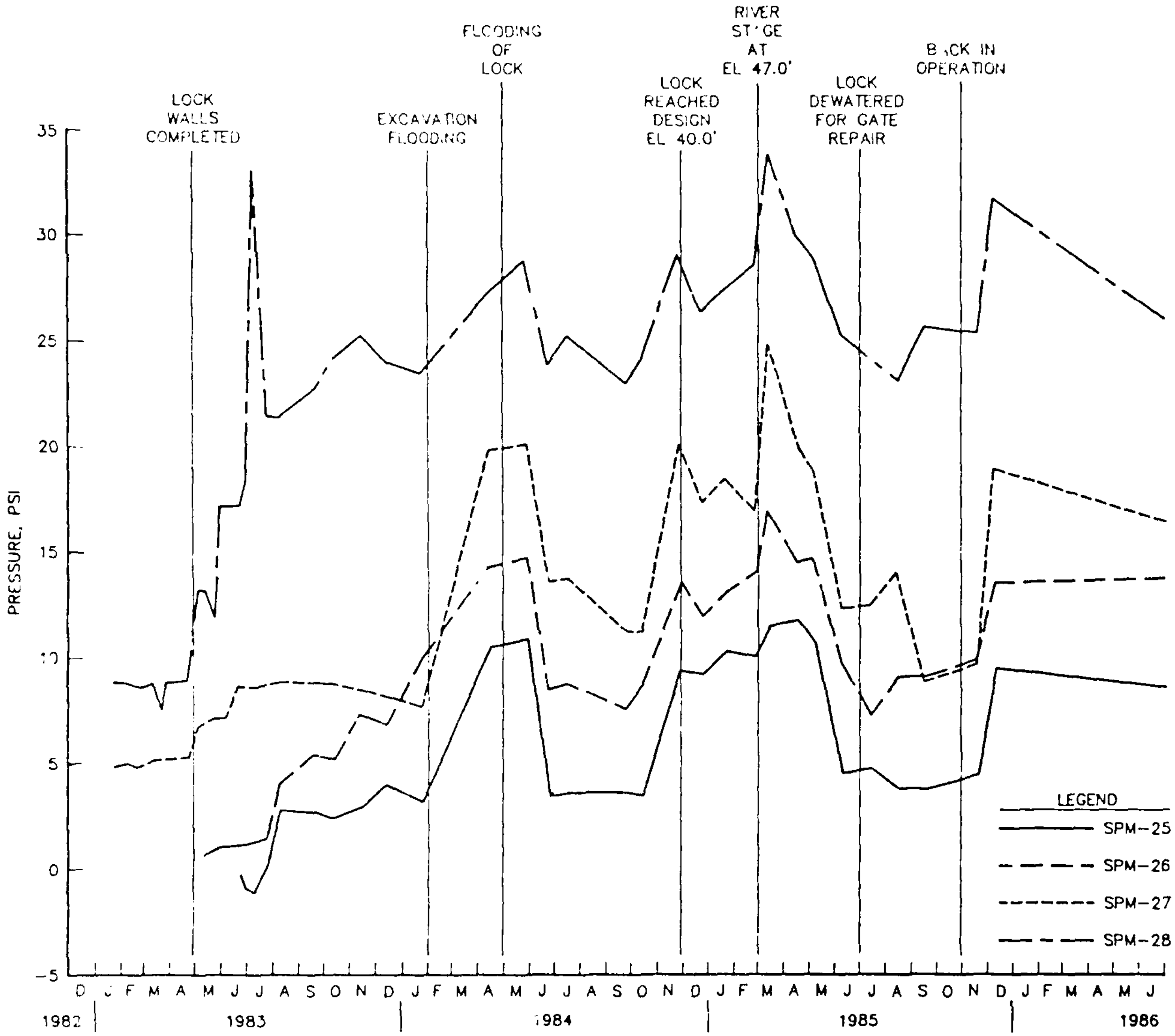


Figure 31. Time plot of earth pressure between backfill and lock wall.

walls and backfill as they were placed. An increase in the pressure devices was observed after excavation flooding, flooding of the lock, and at peak stage of the river. All meters showed a decrease in pressure after operation of the lock in June 1984 and when the lock operation was interrupted in October 1985.

Distribution of lateral earth pressure

55. The coefficient of lateral earth pressure k was computed directly from the observed pressure measurements. The equation used was:

$$k = \frac{P_h}{P_v}$$

P_h is the effective earth pressure in a horizontal direction as mesured by soil stress meters; P_v is the computed effective overburden pressure at the elevation of the meter. The actual earth pressure coefficients for Case II, dewatered condition, varied from 0.26 at the bottom of the culvert to 1.09 at the top. Case III, operating condition, coefficients varied from 0.51 at the bottom to 0.91 at the top. The coefficient value was lower at the bottom, indicating no significant wall displacement and was as expected. The decrease of the coefficient in the operating condition reflects the wall moving toward the lock center line. See Table 2 for comparison of lateral earth pressures for the different cases.

Wall Movements

Lock walls

56. The magnitude of the coefficient of lateral pressure is related to the movement of the wall supporting the backfill. Figure 23 reflects the wall movements during the dewatered condition in September 1985 and operating condition in September 1986. The walls moved as expected, outward for the dewatered condition and inward for the operating condition. The calculated lateral coefficient also verified this movement, see paragraph 55. (As the wall moves toward the lock center line, the coefficients decrease, but as the wall moves toward the backfill, the coefficients increase, Sherman and Trahan, 1968.)

Table 2

Comparison of Lateral Earth Pressures (ksf)

Case	P	P'	P_s	P_s'	P_w	P_w'
I (river)	0.6	0.1	0.6	0.1	-	-
	3.1	1.5	3.1	0.8	-	0.7
	3.7	5.1	3.7	4.0	-	1.1
(land)	0.6	0.3	0.6	0.3	-	-
	3.4	1.7	3.4	1.0	-	0.7
	4.0	3.9	4.0	2.8	-	1.1
II (river)	1.6	1.3	1.6	1.3	-	-
	5.2	1.6	3.4	0.8	1.8	0.8
	5.8	7.2	3.6	5.9	2.2	1.3
(land)	1.6	0.5	1.6	0.5	-	-
	5.3	1.4	3.5	0.6	1.8	0.8
	6.0	4.9	3.8	3.7	2.2	1.2
III (river)	0.6	1.9	0.6	1.0	-	0.9
	4.2	3.3	1.9	0.3	2.3	3.0
	4.7	8.4	2.0	5.0	2.7	3.4
(land)	0.6	1.6	0.6	1.3	-	0.3
	4.3	4.1	2.2	1.2	2.1	2.9
	5.0	5.6	2.5	2.2	2.5	3.4

	Predicted	Observed
Total pressure	P	P'
Water pressure	P_w	P_w'
Soil pressure	P_s	P_s'

57. Inclinometers numbers 1 through 4 were installed as part of the instrumentation program. Numbers 5 and 6 were added later when there was a concern over the movements of the floodwall monoliths east of the dam. The relative positions and the orientations of the inclinometers are shown in figure 32, along with directional movements. The responsibility for reading the inclinometers changed hands so many times, each party reading them differently, that data interpretation is difficult. Initial readings were taken on 23 January 1985; as recommended by jobsite personnel, all prior readings were ignored for this analysis.

58. Inclinometer number 3 was reported to be broken. The inclinometer readings for the lock are summarized in the "Data Summary", figures B-15 through B-20 and B-102 through B-103.

59. There was no consistency in the direction of the wall movements based on the inclinometer data. It was also impossible to establish the direction based on the chaining measurement across the lock chamber due to insufficient data.

Floodwalls

60. Figure 33 shows the floodwall movements for 19 September 1985 and 9 September 1986 as measured by the reference bolts. The floodwall monolith that was of concern was monitored by inclinometers numbers 5 and 6 (refer to figure 30 for the locations). The summarized data are shown in figures B-405 through B-410 in the "Data Summary" report. A comparison was made by isolating the inclinometer data for the same time frame (the directions of the floodwall movements are shown in figure 32). The data from the reference bolts (RB-62 and RB-63) and the inclinometers (INC-5 and INC-6) showed floodwall F-3 movement in the downstream direction. The maximum movement east of the dam was 0.016 feet toward the downstream direction between F-7 and F-8. Maximum movement of 0.040 feet downstream on the west side occurred between the dam and F-10. East of the lock, maximum movement was 0.015 feet upstream between F-21 and F-22.

61. Inclinometer number 5 indicates an abrupt movement near the bottom; a visual inspection should be sufficient to verify structural distress or a twist in the inclinometer casing.

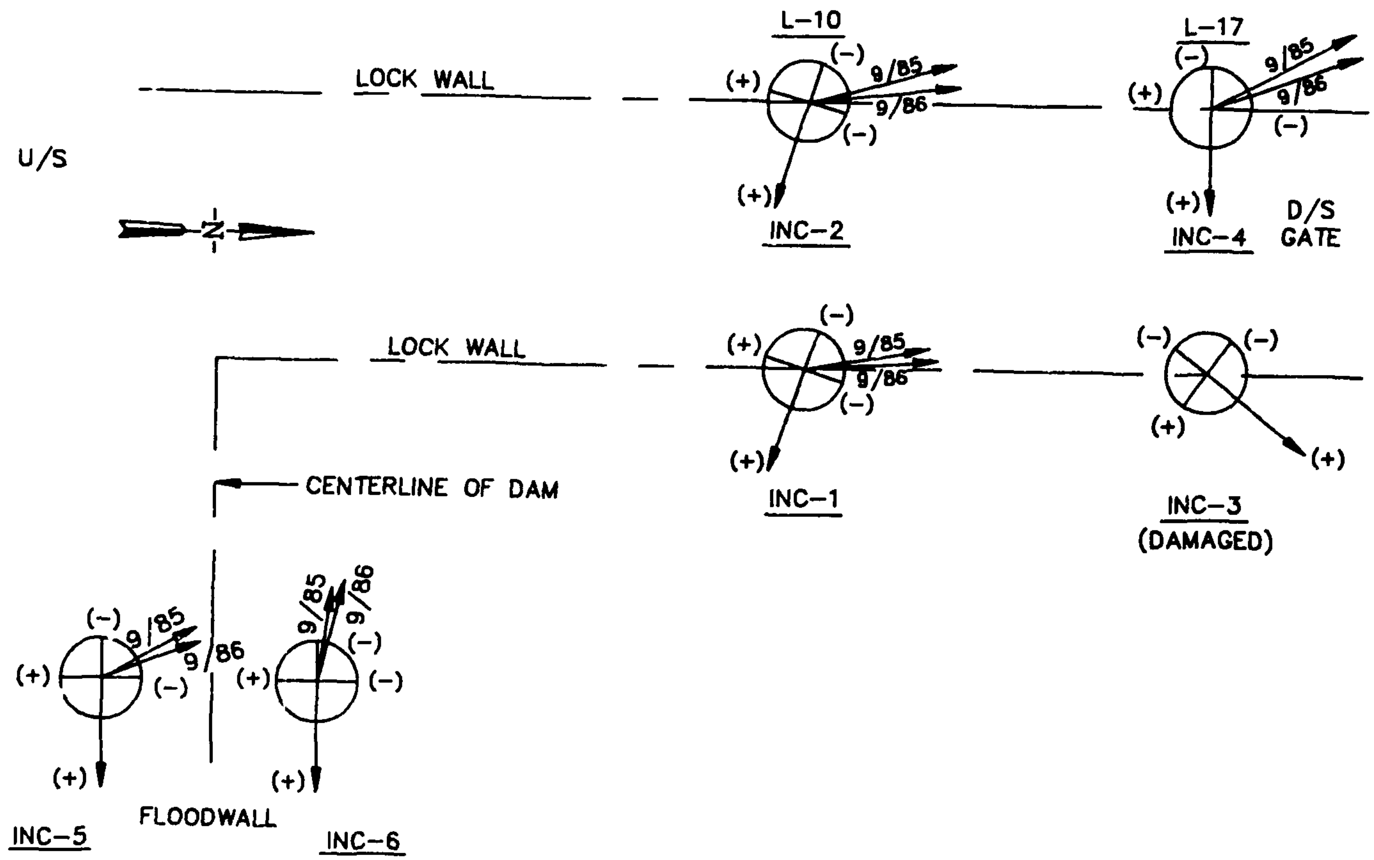


Figure 32. Directions of wall movement as indicated by inclinometers.

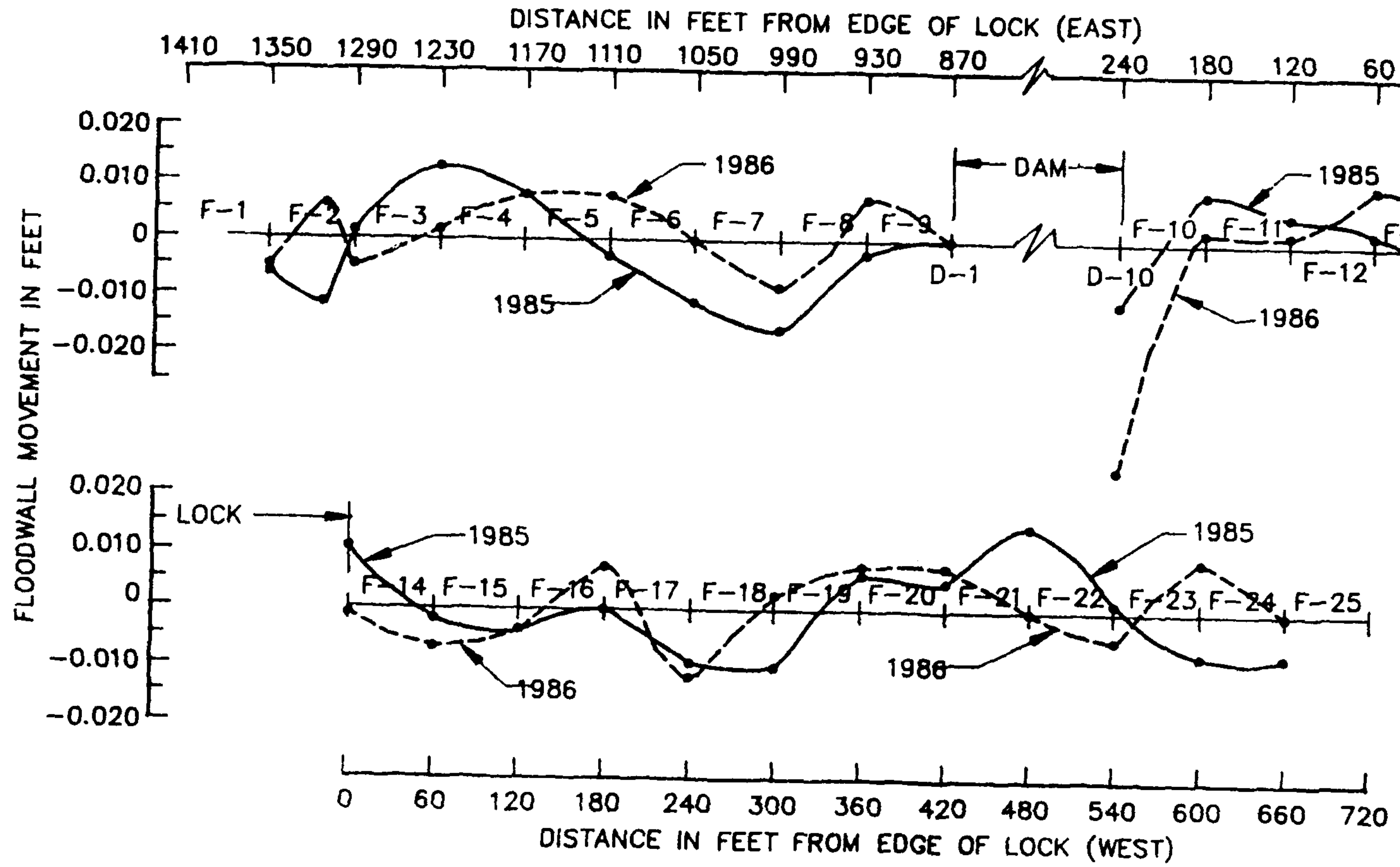


Figure 33. Floodwall movements for September 1985 and 1986.

Comparison of Observed and Predicted Earth Pressures

62. Table 2 contains a summary of the comparison between observed and predicted lateral earth pressures for the construction, dewatered and operating cases of monolith L-10. Refer to figures 6, 7 and 8 for the predicted values for all three cases. The observed pressure distributions are in reasonably fair agreement with those assumed in design, considering the different factors. Case I assumed that there was no water pressure since the water table was below the foundation slab. Piezometers observed some water pressures for the actual condition. The observed water elevation with comparable conditions to Case III was lower than the assumed cases. The other factor in obtaining a realistic comparison is the accuracy of the instrumentation. Local variations in soil condition, temperature effect on the soil pressure cells, the stiffness of the cells, the irregularity of the surface of the structure and the laboratory calibration are just some of the variables, in addition to the installation and observation techniques which influence accuracy.

63. The maximum pressures were consistently observed at the lower riverside of the lock for all three cases. The soil pressure was higher than predicted in some cases while the water pressures were close except for Case III. The nonconforming pressures were recorded at SPM-37. The time history plot for this instrument and other associated meters show a jump of almost 25 psi in figure B-67 in the "Data Summary" report. The increase reflects the excavation flooding in early 1984.

PART VII: FOUNDATION BASE PRESSURE AND UPLIFT

Observed foundation base pressures

64. The time plots of observed base pressures shown in figures B-66 to B-79, B-94 to B-99, and B-121 to B-131 in the "Data Summary" show that the stress meters were responding to the applied loads. Figure 34 shows a steady increase in the pressure from the beginning of lock wall construction to the time of completion at the end of April 1983. The effects of the first three lifts can be identified by the peaks in the curves. Other foundation soil pressure meters reflect the same pattern of responses. Refer to figure 35 for locations of soil pressure meters and piezometers in lock monolith L-10.

Observed base pressures

65. Table 3 contains base pressures for the lock observed during construction, dewatered and operating conditions, Cases I, II and III, respectively. Figure 36 shows the base pressures for various conditions in monolith L-10. The comparison shows an ascending pattern of base pressure going from the stages of construction to dewatered to operating.

66. The base pressure under the dam was in close agreement with the predicted values (see figures 37 and 38 for locations of instrumentation and diagram of predicted values, respectively). Comparisons for construction and operating cases are listed below.

Cases	Base Pressures (ksf)	
	(Predicted)	(Observed)
Construction	2.6	1.9
Operating	5.1	4.0

Comparison of observed and predicted uplift pressures

67. The uplift pressures for various conditions are plotted in figure 39 for lock monolith L-10. Note that water pressure existed at the actual construction time used in the analysis. The observed values for the dewatered

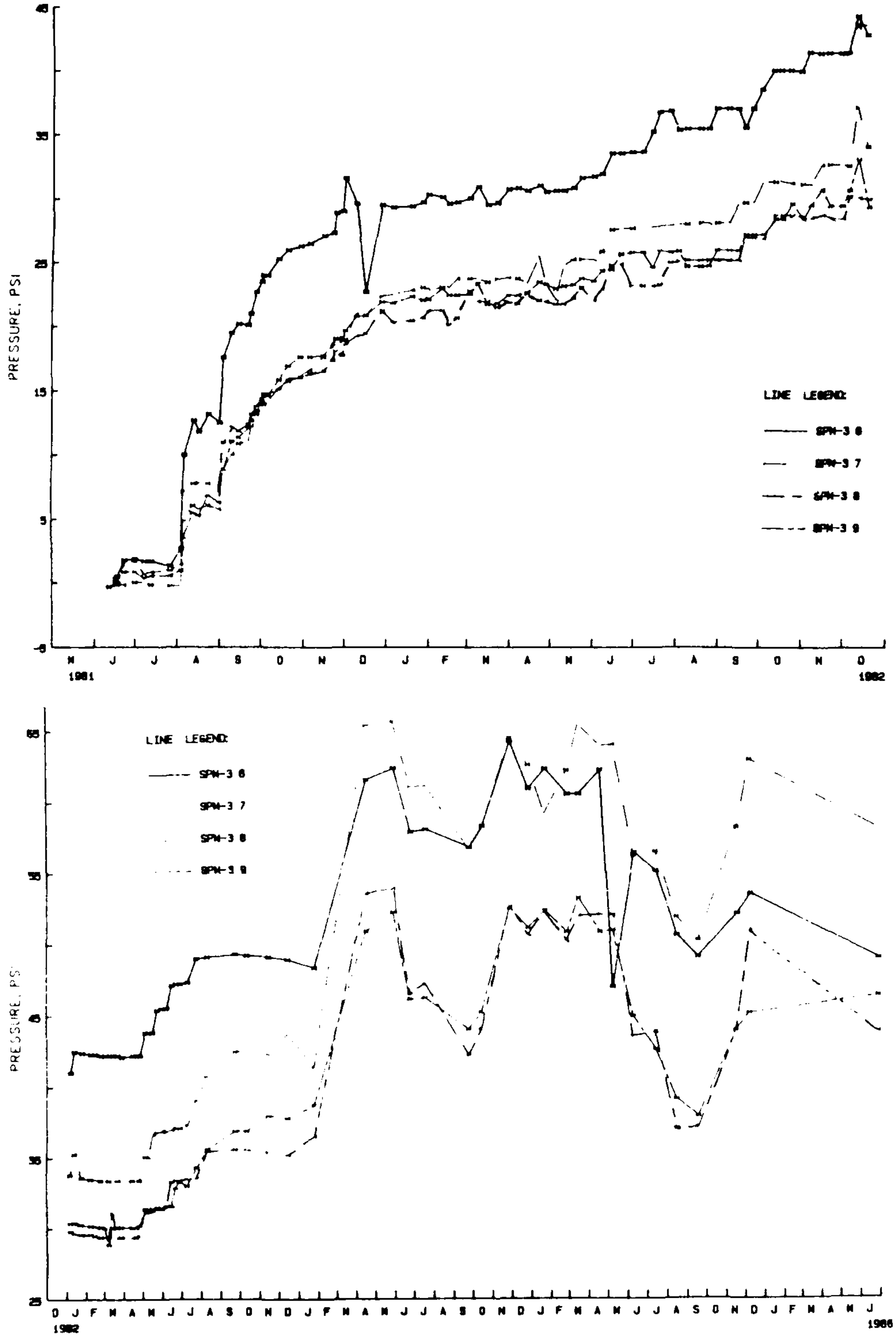
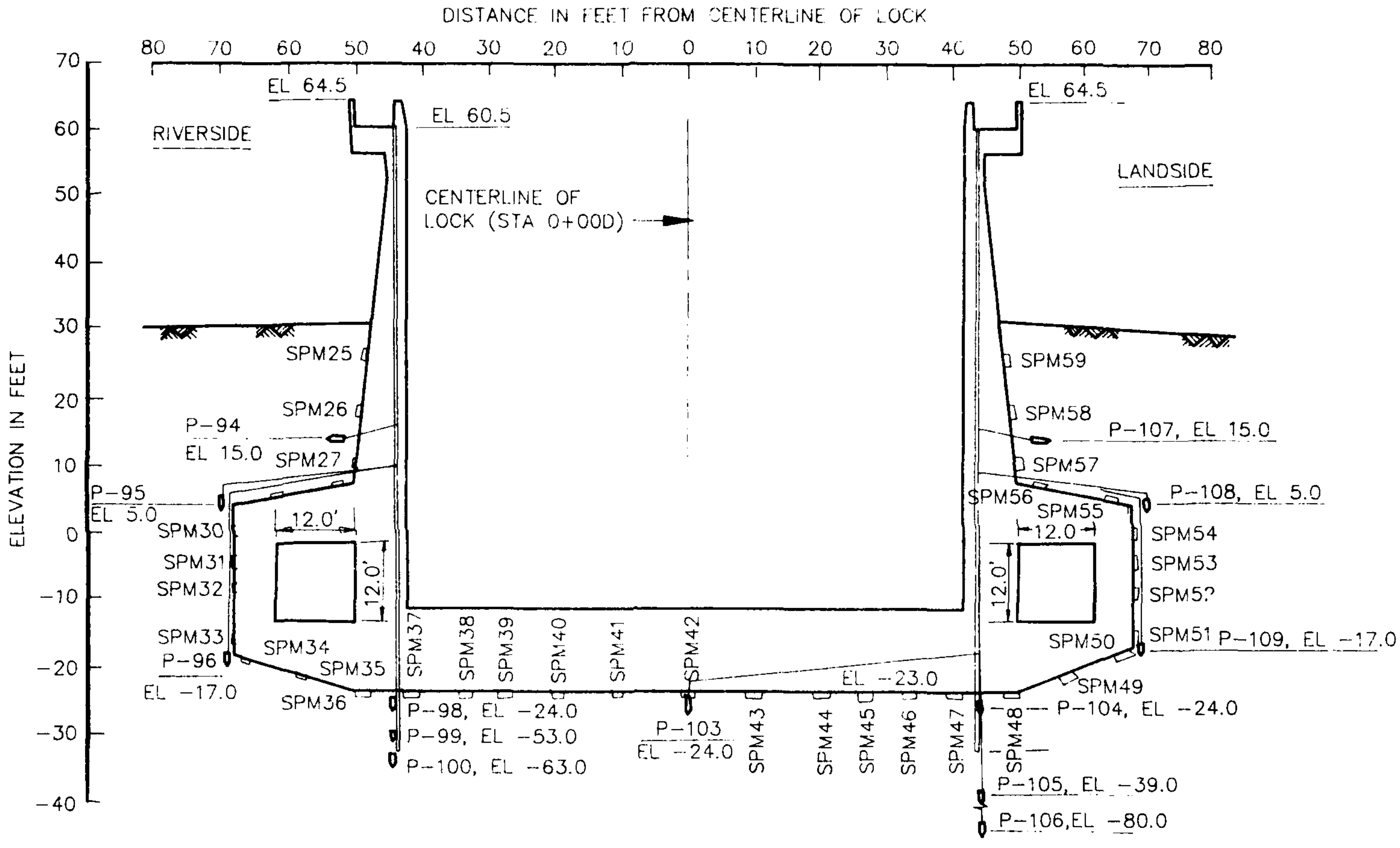


Figure 34. Typical time plot for soil pressure meters from 1981 to 1986.



SECTION A

Figure 35. Location for soil pressure meters in lock monolith L-10.

Table 3

Comparison of Base Pressures (ksf)

SPM#	Case I	Case II	Case III
34	.96	1.25	3.07
35	1.09	1.15	3.17
36	6.31	7.06	7.69
37	5.05	7.23	8.35
38	4.52	5.34	6.31
39	4.52	5.44	6.67
40	4.39	4.25	5.69
41	2.29	2.56	3.30
42	3.33	4.12	5.37
43	4.26	5.11	6.13
44	5.98	5.64	7.72
45	4.79	6.51	7.06
46	4.78	5.05	5.71
47	3.87	4.89	5.57
48	5.17	5.96	7.40
49	2.72	1.81	3.90
50	2.97	2.46	4.13

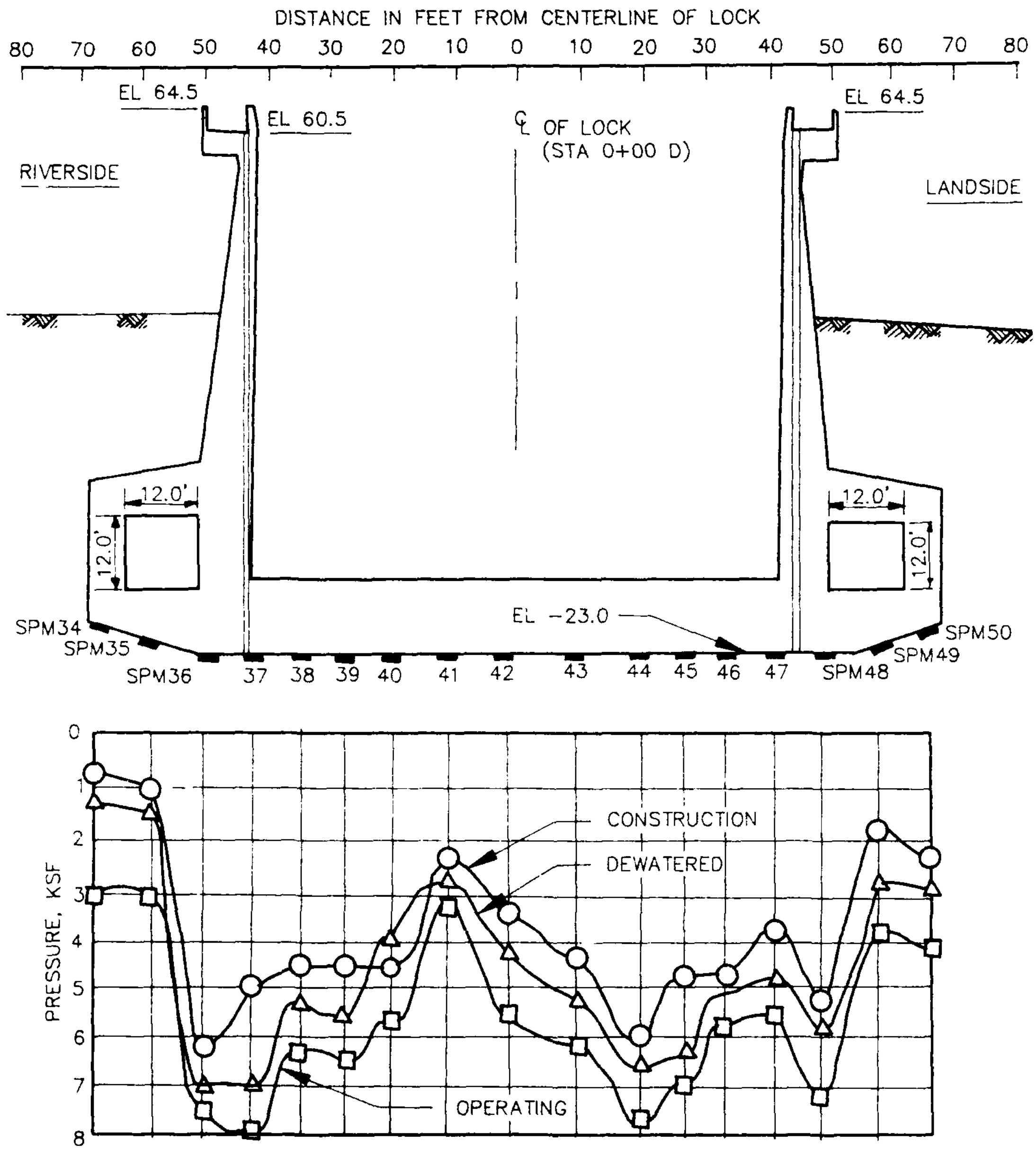


Figure 36. Base pressures for various conditions in lock monolith L-10.

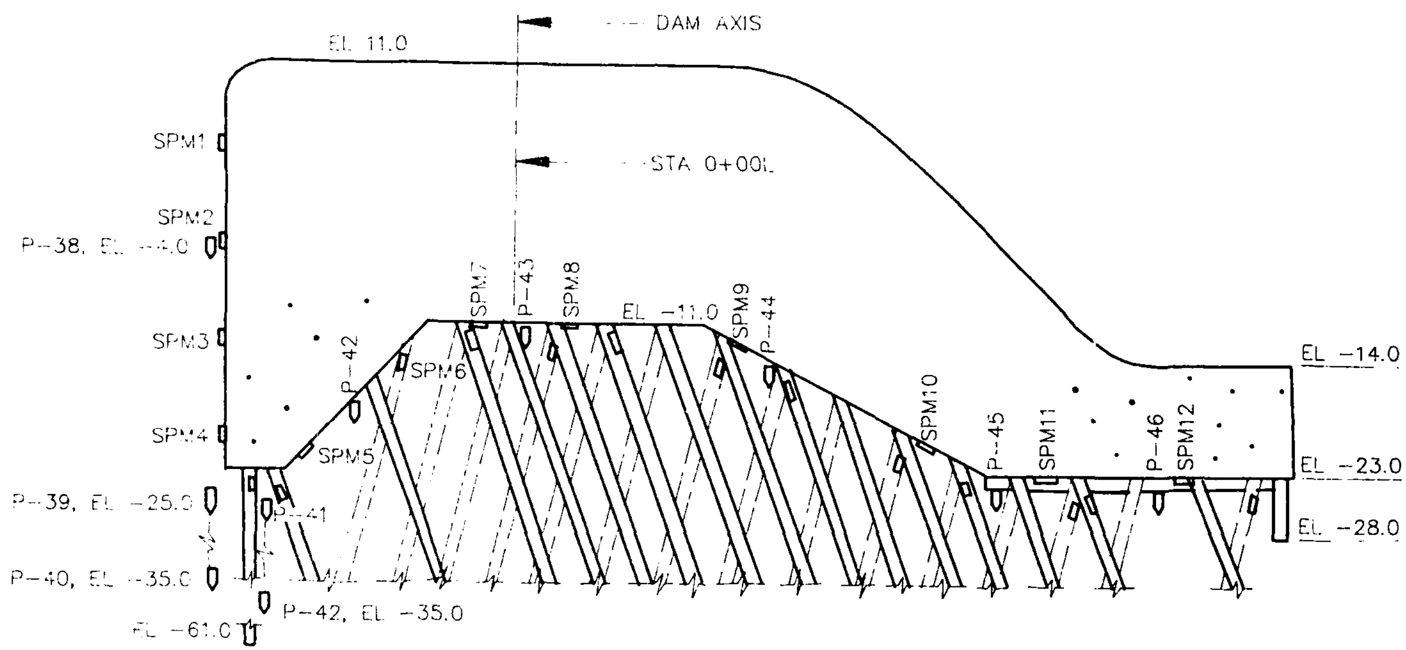
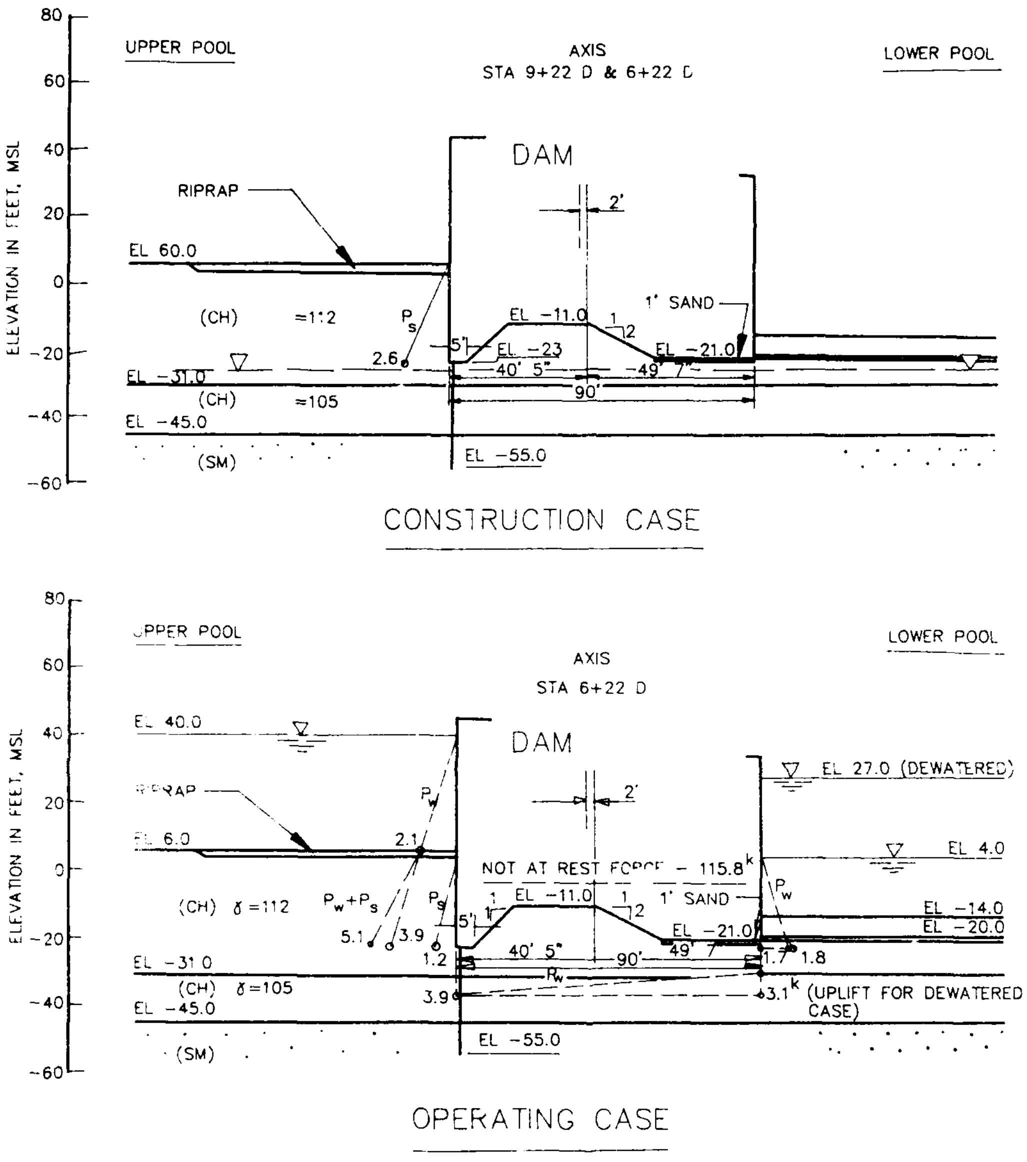


Figure 37. Location for soil pressure meters and piezometers in dam section D-5.



SOIL PARAMETERS		
TYPE	LB/CU FT	K_o
CLAY BACKFILL	115.0	0.8
SAND BACKFILL	122.5	0.5
CH	112.5	0.8
ML	117.5	0.6

Figure 38. Predicted earth and water pressures for the dam.

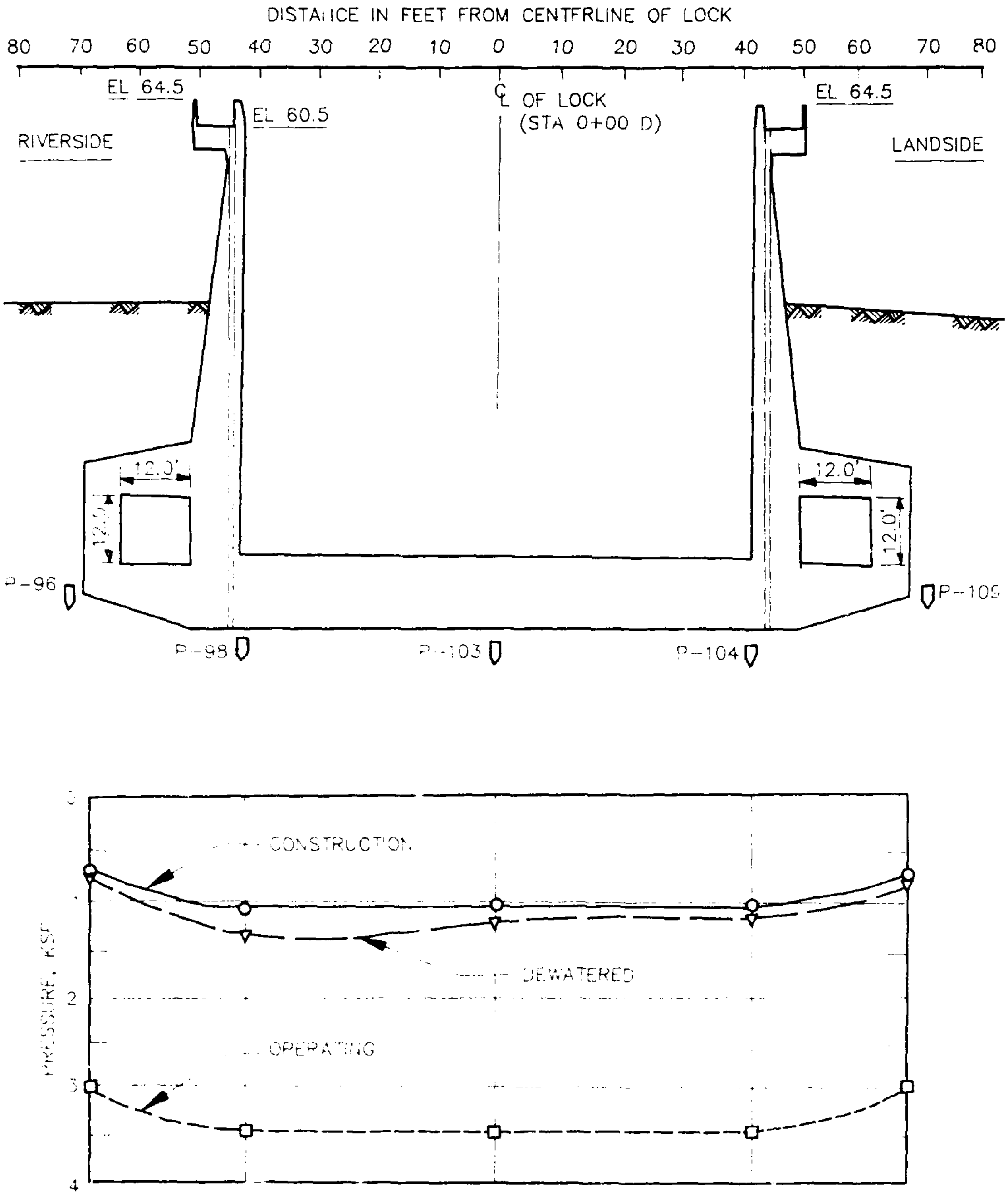


Figure 39. Uplift pressures for construction, dewatered and operating conditions in lock monolith L-10.

case were lower than predicted pressures. The results for the operating case were the opposite; the observed values were higher than predicted. This is explained by the differences in the water elevation for both cases. The pressure differences in the dewatered case was 0.29 ksf and 0.72 ksf for the operating case. The summary for the uplift pressures for lock monolith L-10 is listed in Table 4.

68. The uplift pressures for the dam are lower than predicted. the results are listed below.

Cases	Uplift Pressures (ksf)	
	(Predicted)	(Observed)
Construction	-	4.4
Operating	3.9	3.3

Table 4

Comparison of Uplift Pressures (ksf) in L-10

Case I (construction)		Case II (dewatered)		Case III (operating)	
P_w	P_w'	P_w	P_w'	P_w	P_w'
-	0.7	1.8	0.7	2.3	3.0
-	1.1	2.2	1.2	2.7	3.4
-	1.1	2.2	1.2	2.5	3.4
-	0.7	1.8	0.7	2.1	3.0

P_w predicted water pressure

P_w' observed water pressure

PART VIII: BENDING MOMENTS IN BASE SLAB AND WALLS

69. An additional evaluation of the instrumentation data was conducted by comparing moments and deflections from CUFRAM analyses with moments and deflections calculated using RSG (resteel strain gage) data in the base slab. Also, CUFRAM moments in the walls and culverts were compared with moments found using RSG data. In the CUFRAM analyses, soil pressure meter data were used to estimate external loads. Instrumentation data obtained for monolith L-10 on 22 September 1983 were used in the analyses. These data were selected because most RSG data appear to be valid and because loading corresponds to a Case II' condition as considered by "Analysis Of Data From Instrumentation Program, Port Allen Lock" (Sherman and Trahan, 1968). That analysis defined Case II' as the loading applied to a complete structure with backfill in place, but with no water in the lock. That analysis also modeled a maximum operating condition (Case III'), in which the lock was assumed to be in operation with water at an elevation of 49 feet in the chamber. Because no data were available indicating chamber water elevations, no similar analysis was conducted in the current study for Lock and Dam No. 1.

Moments Based on Applied Loads

Moments in the base slab

70. Moments in the base slab were calculated using the observed soil pressures as loadings in CUFRAM. The observed total pressures are shown in figure 40 for the base, riverside wall and landside wall. The CUFRAM input pressures are shown in figure 41. CUFRAM allows the user to enforce vertical equilibrium of the weight of the structure and vertical base pressures by taking unbalanced forces as frictional forces in the walls or adjusting the base pressures. Both options were used in the current study. Figure 41 shows the base pressures distribution obtained by enforcing vertical equilibrium by considering the unbalanced forces as frictional forces. This figure is almost identical to the one with the base pressure distribution obtained by considering a redistribution of base pressure for vertical equilibrium. The small difference is largely because the vertical force imbalance was only 5 kips.

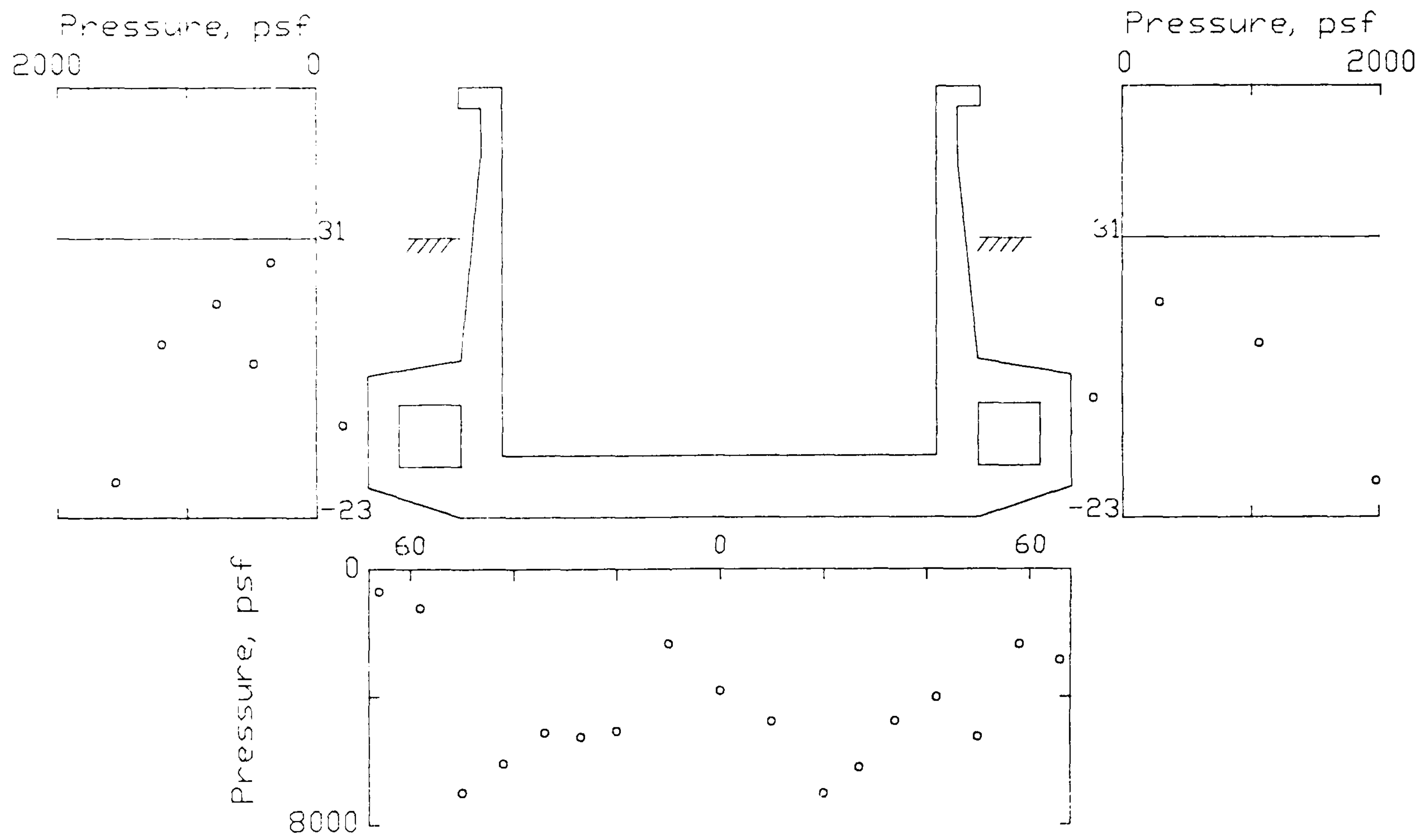


Figure 40. Observed soil pressure meter data for 22 September 1983.

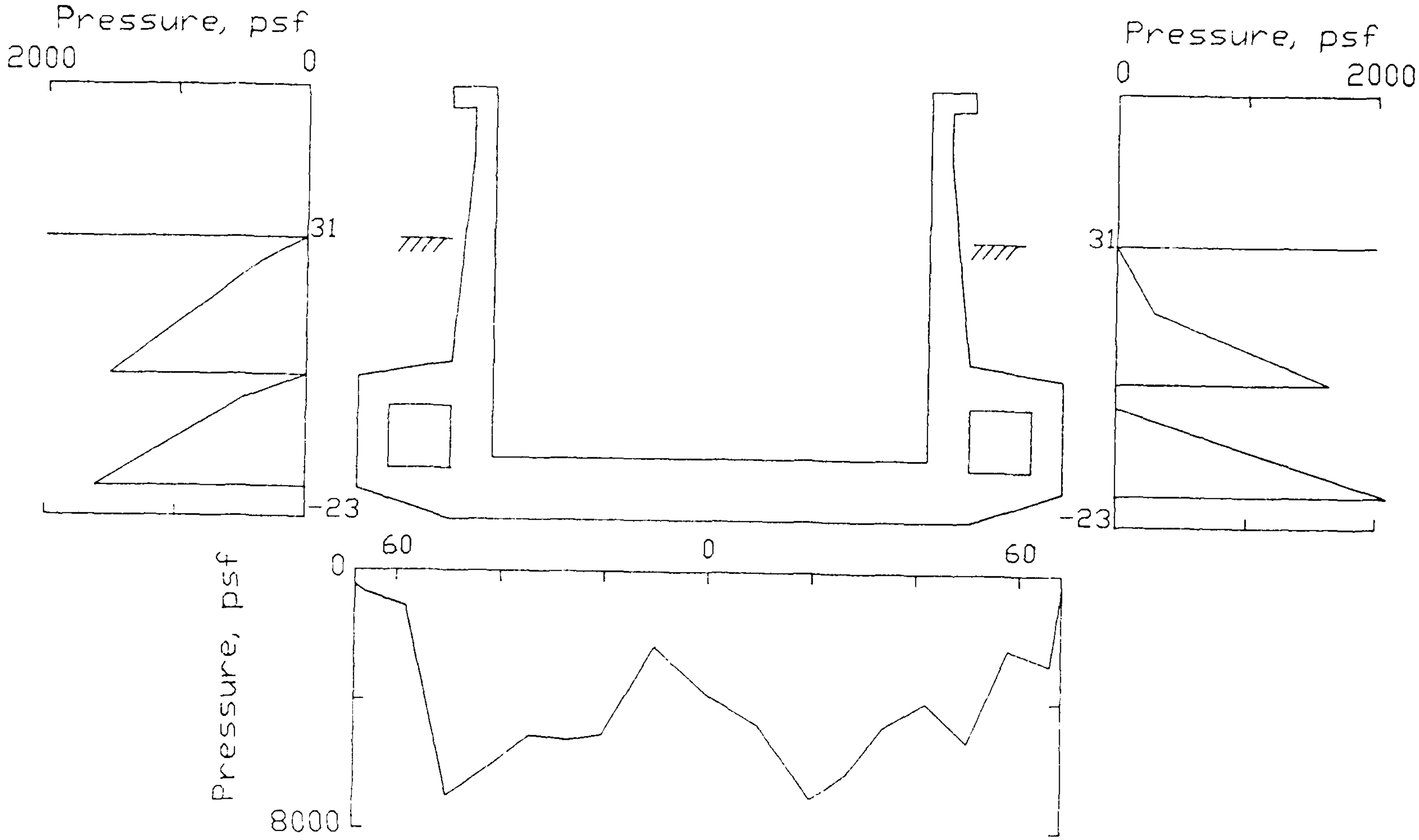


Figure 41. CUFRAM input (with friction).

71. In both CUFRAM analyses, the lock chamber was assumed to be completely dewatered. Lateral pressure distribution on the walls and culverts were based on observed pressure data (figures 40 and 41). Vertical pressures were assumed to be linear functions of the soil depth. A unit weight of 120 pcf was used.

72. Moments computed using the above assumptions are shown in figure 42. Comparison of the graphs show that the two methods of enforcing equilibrium provided similar results. The maximum calculated moments (at the centerline of the base slab) were 2537 ft-kips and 2337 ft-kips for analyses using wall friction and adjustment of pressures, respectively.

73. For comparison with the results obtained using observed base pressures, a CUFRAM analysis was made using a uniform base pressure distribution (figure 43). All lateral pressure distributions were based on observed data and matched those from the other CUFRAM analyses. The base pressure distribution was automatically adjusted to maintain vertical equilibrium. The calculated moments, shown in figure 42, are significantly lower than those obtained using the observed base pressures. These differences are because the base pressures below the culverts for the measured base pressure are much lower than the assumed uniform pressure. Load is therefore concentrated in the base slab between the lock walls resulting in large increases in moment relative to those obtained using the uniform distribution. A similar set of analyses on the Port Allen lock showed much better agreement between observed and assumed pressures. While the Lock and Dam No. 1 data show rapid drops in pressure in the culvert regions, the Port Allen lock data show relatively high pressures over the full width of the culverts.

Moments in the walls

74. Moments for the walls obtained in the CUFRAM analyses are shown in figure 42. Since the three analyses used the same lateral earth pressures, all three analyses provided the same moment distributions. The maximum moment, calculated at an elevation of 7.5 feet in the riverside wall, was 172 ft-kips (tension on the backfill side of the wall).

Moments in the culverts

75. The maximum moments obtained for the culvert members using CUFRAM are shown in figure 44. Although the two analyses based on observed pressures show good agreement, the analysis based on the uniform base pressures shows

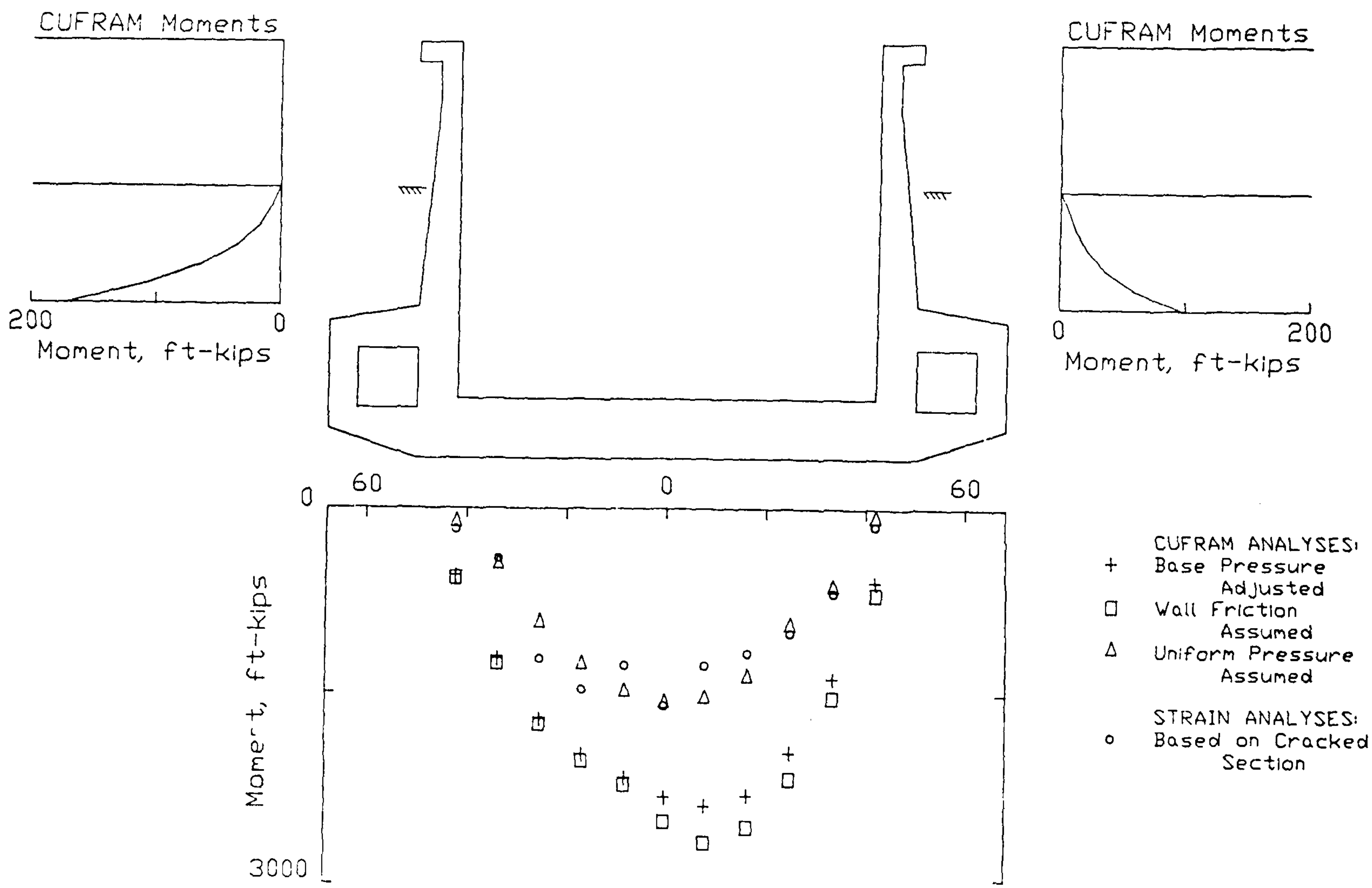


Figure 42. Comparison of moments.

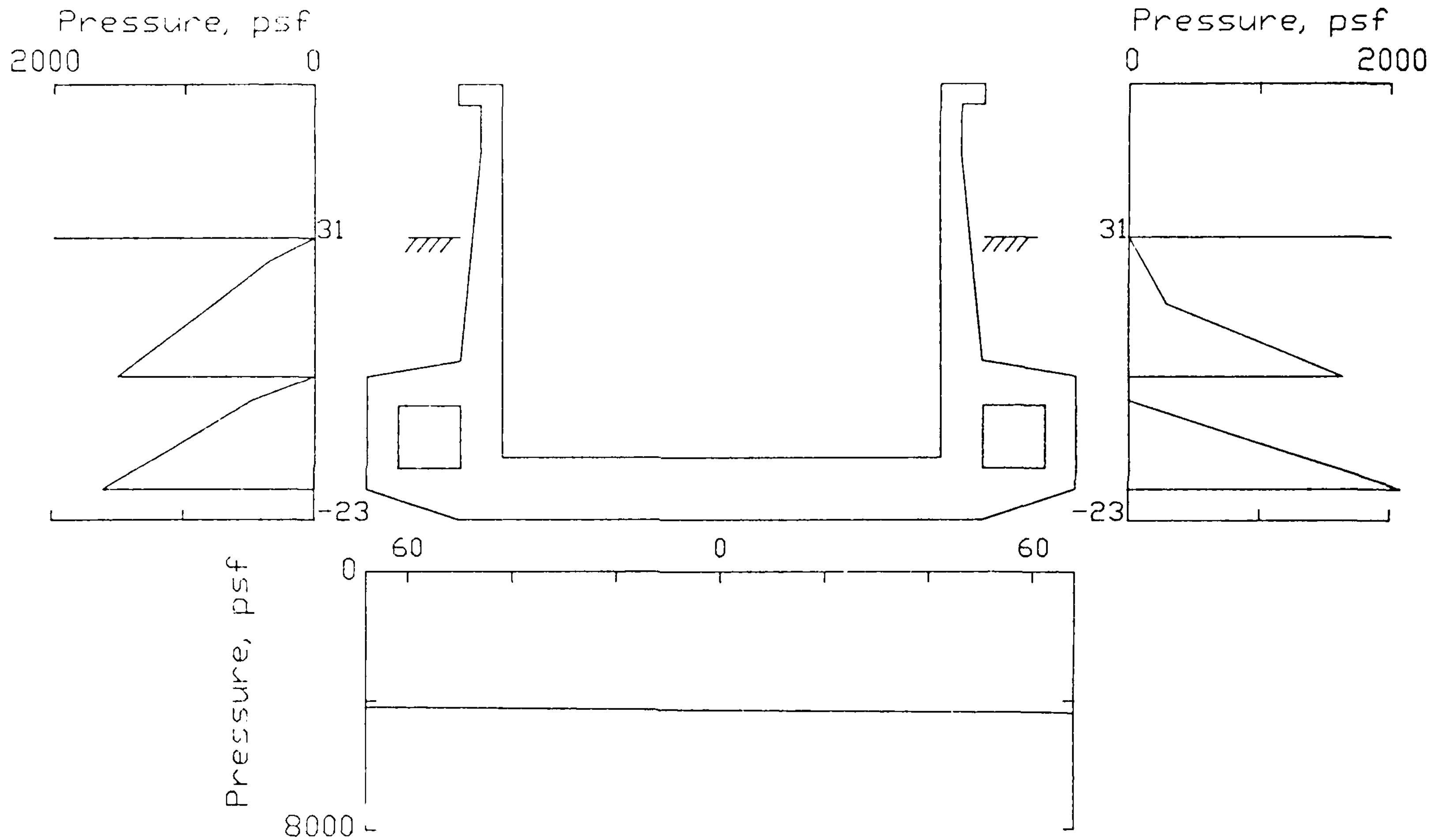
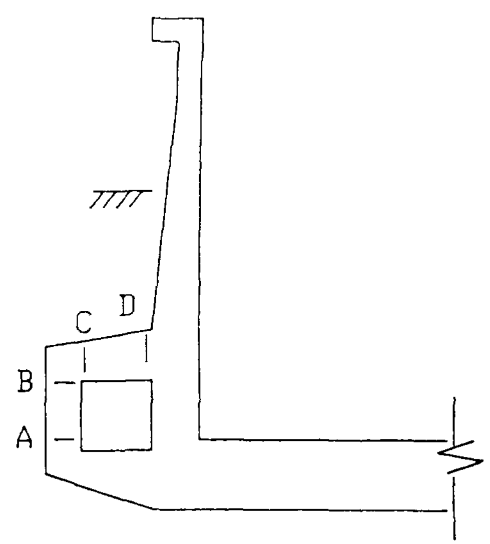


Figure 43. Uniform base pressure distribution.

	Moment, ft-kips		
Section	Redistr. Pressure	Wall Friction	Uniform Pressure
A	-54.8	-55.9	-18.8
B	109.0	111.0	45.1
C	82.8	84.2	34.0
D	-186.8	-186.7	-186.7



Note: Positive Moment Produces Compression on the Side Toward the Chamber Centerline

Figure 44. CUFRAM moments in culvert members.

much lower moments in three of the four locations. This difference is reasonable since the large vertical pressures assumed on the top of the culvert (resulting from the assumed soil unit weight of 120 pcf) are balanced by the uniform base pressure distribution, but is not balanced by the other two base pressure distributions.

Deflections Based on Calculated Moments

76. The Port Allen lock report verified calculated moments by comparing observed deflections with those estimated from an analysis of the structure. A similar verification was attempted in the current study; however, it was not possible to do so.

77. The deflection data obtained from Lock and Dam No. 1 indicate that, in the regions near the walls, the base slab deflected upward as the walls were constructed and as the backfill was completed. Because this is contrary to the observed behavior of similar structures, the deflection data were considered to be invalid. Deflection data extrapolated from the CUFRAM analysis and from the resteel strain gage (RSG) data were, however, compared. A best fit of the moment values obtained in the CUFRAM analysis was used to calculate deflections. Also, a best fit of the curvatures obtained from the RSG data was used to calculate deflections.

78. The best fit of the moment data obtained from the CUFRAM analysis which included redistribution of the base pressures is given by:

$$M = 2594 - 4.942x - 1.089x^2$$

where

M = moment in ft-kips

x = horizontal distance from the centerline of the slab in feet.

Since $y'' = M/EI$, the deflected shape can be determined upon establishment of boundary conditions. For this case, the deflection slope of the deflection curve at the slab centerline was assumed to be zero. Also, E was assumed to be 4030 ksi, and I was assumed to be the gross moment of inertia of the slab.

79. A total of eleven pairs of RSG's were located in the base slab in monolith L-10. A summary of the strain readings in the base slab is provided in figure 45. The total distance between the top and bottom gages of each pair was 10 feet. The curvature of the slab at each gage location was therefore estimated by dividing the difference in the two strain readings by 10 feet. A best fit of the curvature data is given by:

$$y'' = 45.9483 - 0.01441x - 0.02136x^2$$

The deflection of any point relative to the centerline was found by integrating the curvature equation and by using the same boundary conditions as were used for the deflection calculations based on moment.

80. The deflections computed using the CUFRAM moments and the RSG data are shown in figure 46. The two curves show relatively good agreement. It should be noted that the tensile strains measured in the top of the base slab (figure 45) indicate that the slab was cracked over a portion of the width. Actual deflections were, therefore, probably much larger than those indicated by the above analyses.

Moments from Internal Stresses

Internal strains

81. As mentioned above, the base slab was probably cracked in service. Moments based on the strain information were therefore obtained using a cracked section modulus. Moment calculations were based on the following assumptions:

a. Strain compatibility exists between the steel and the concrete.

b. Compression stress in the concrete is given by a parabolic stress-strain relationship.

c. The modulus of elasticity for the steel is 29000 ksi.

82. Base slab moments calculated from the strain data are shown in figure 42. Interestingly, the moment curve based on the strain data is very close to that obtained in the CUFRAM analysis based on a uniform base pressure distribution. The analyses based on the observed soil pressures are nearly twice as large as those found using the RSG data.

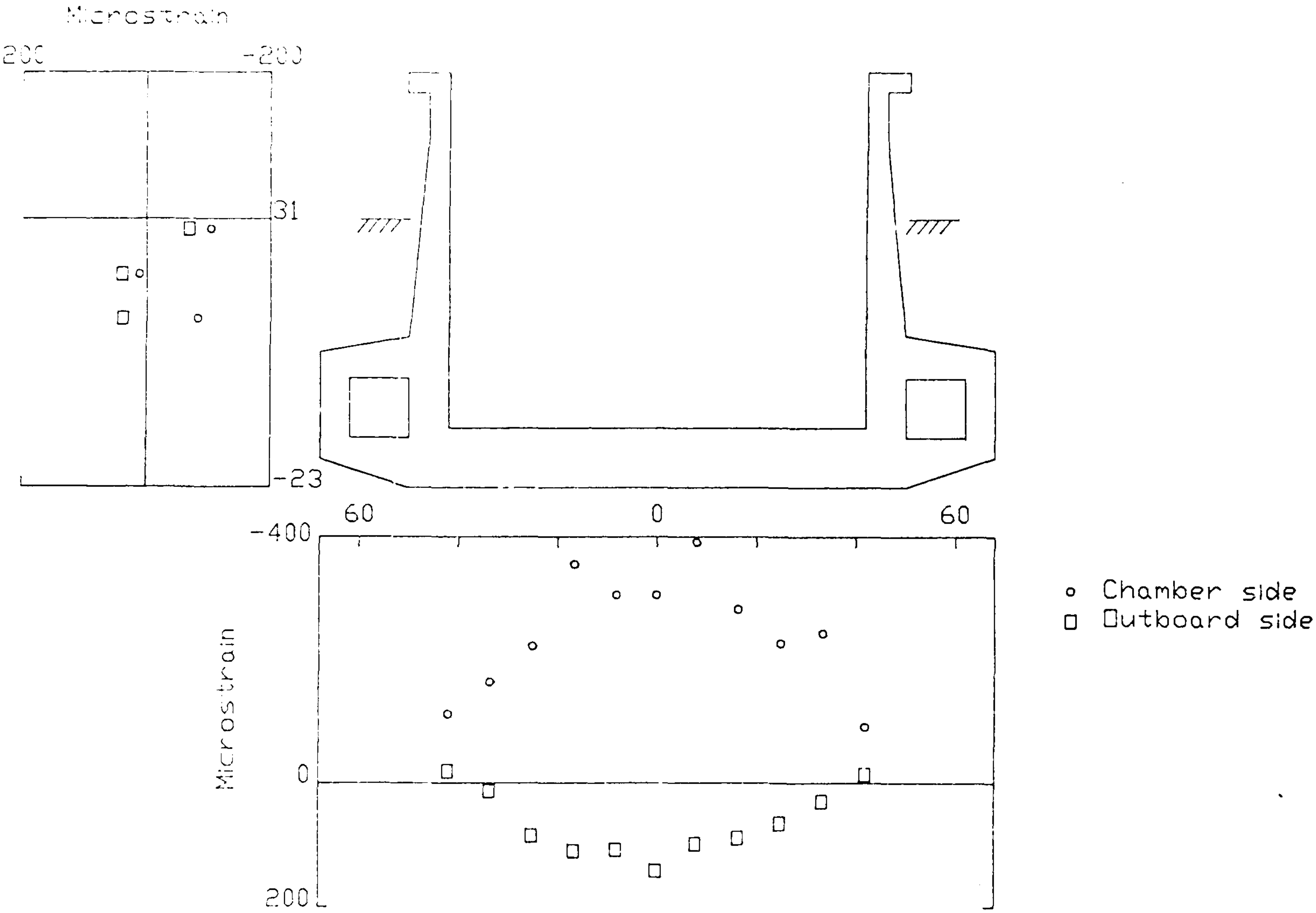


Figure 45. Strain readings in base slab and wall.

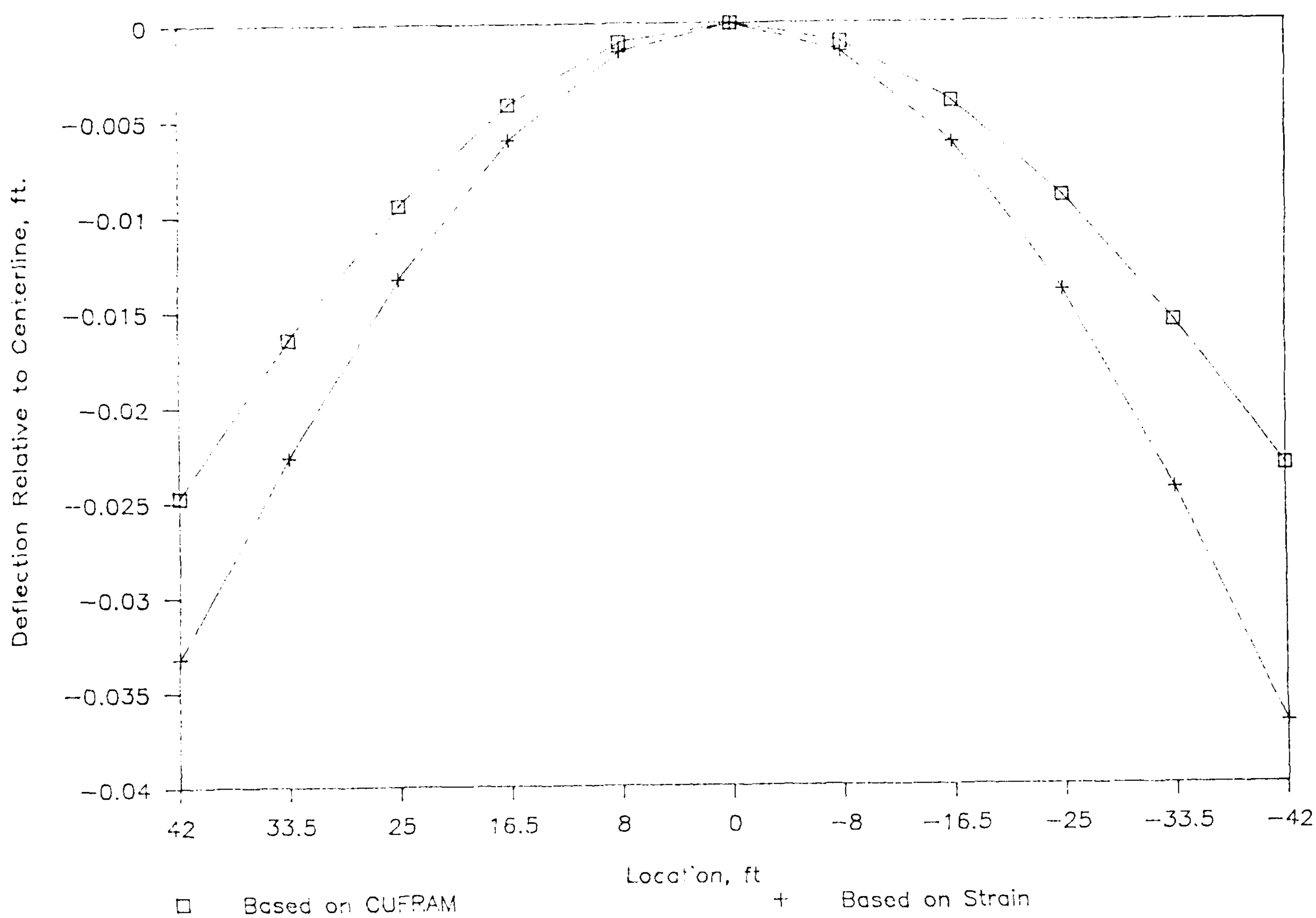


Figure 46. Calculated deflections in base slab (zero centerline deflection assumed).

83. Insufficient data makes it impossible to develop full moment curves based on RSG data for the walls and culverts. However, comparisons with CUFRAM moments can be made at selected points. Also, comparisons can be made between the RSG analyses and the three loadings applied to the lock structure using CUFRAM.

84. For the walls, the moment calculated at an elevation of 11 feet using RSG data is 378 ft-kips, with compression on the chamber side of the wall. The corresponding CUFRAM moment is 117 kips, also with compression on the chamber side. It should be noted that the RSG data do not show reasonable continuity through the height of the wall structure (figure 45).

85. For the culverts, moments from RSG data and the corresponding moments from the lock analyses are shown in Table 5. The tabulated values show that not only do the magnitudes of the moments vary widely, but the sense of the moments from the RSG data matches the sense of the moments from the CUFRAM analysis in only half of the cases. The members in the culverts are all relatively short. Because the members are in regions of high stress concentration and discontinuity, the analyses of the strain data and the linear analyses using beam elements (CUFRAM) are probably not reliable.

Table 5

Moments Comparison for the Culverts

Near Section (see fig.46)	RSG Moment	CUFRAM Moments Redistri. Pressure	Wall Friction	Uniform Pressure
A	11.8	76.5	76.7	34.5
B	172.0	-20.7	-21.0	-9.6
C	221.0	82.8	84.2	34.0
D	362.0	-186.8	-186.7	-186.7

PART IX: ANALYSIS OF PILE DATA

86. Stress time history data were analyzed for three piles under the dam. The piles consisted of one "H" pile below the spillway and two sheet piles above. The objective was to determine if the high water condition during the flood of March 1985 through April 1985 or the low water conditions during the summer of 1985 had significant loading effects on the piles. These time periods correspond to silt build up and dewatering of the lock, respectively.

87. Structural responses of these piles were reviewed for correlation to hydrostatic data of the river's surface elevation. Locations of these piles and general instrumentation layout are given in table 6. The "H" pile gages are designated SG-24, (pile number 1870) with the pile located below the spillway in the center of the dam as shown in figure 47. Specific gage locations on the "H" pile are shown in figure 48 with a total of five vertical gages that measure axial strain. These gages are located 6 inches below the dam's concrete bottom. The PZ-38 sheet pile was instrumented in two primary locations (thus two piles for this discussion) as shown in figure 47. Gages were installed as shown in figure 49 with specific elevations given in table 6. One sheet pile had gage designations from SG-25 through SG-35 and the other from SG-36 through SG-46 as given in table 6.

88. Hydrostatic data for the river elevations versus time from September 1984 through December 1986 are shown in figure 10. Flooding during March and April 1985 is indicated with high water above and below the dam. A low water condition existed from July through October 1985 with the upstream pool level below normal during September and October. Other low water conditions occurred during the fall of 1984 and summer of 1986 and a flood occurred during December of 1985.

89. Recorded "H" pile stresses are shown in figure 50. The stress wave forms correlate well with downstream elevations shown in figure 10. Typically, the stress decreases (becomes more compressive) as the river rises, indicating the pile is carrying more axial load. This can be clearly seen in figure 50, for the flood of March 1985 through April 1985, with the downward slope of all five stress readings indicating additional axial load being

Table 6

Gage Designations and Locations for PZ-38 Sheet Pile

Designation	Monolith	Dam Station	Lock Station	Elevation
SG-25	D-5	6+70.5D	-0+23L	-20.0
SG-26				-23.0
SG-27				-26.0
SG-28				-29.0
SG-29				-32.0
SG-30				-35.0
SG-31				-38.0
SG-32				-41.0
SG-33				-46.0
SG-34				-51.0
SG-35				-56.0
SG-36	D-5	6+51.0D	-0+23L	-20.0
SG-37				-23.0
SG-38				-26.0
SG-39				-29.0
SG-40				-32.0
SG-41				-35.0
SG-42				-38.0
SG-43				-41.0
SG-44				-46.0
SG-45				-51.0
SG-46				-56.0

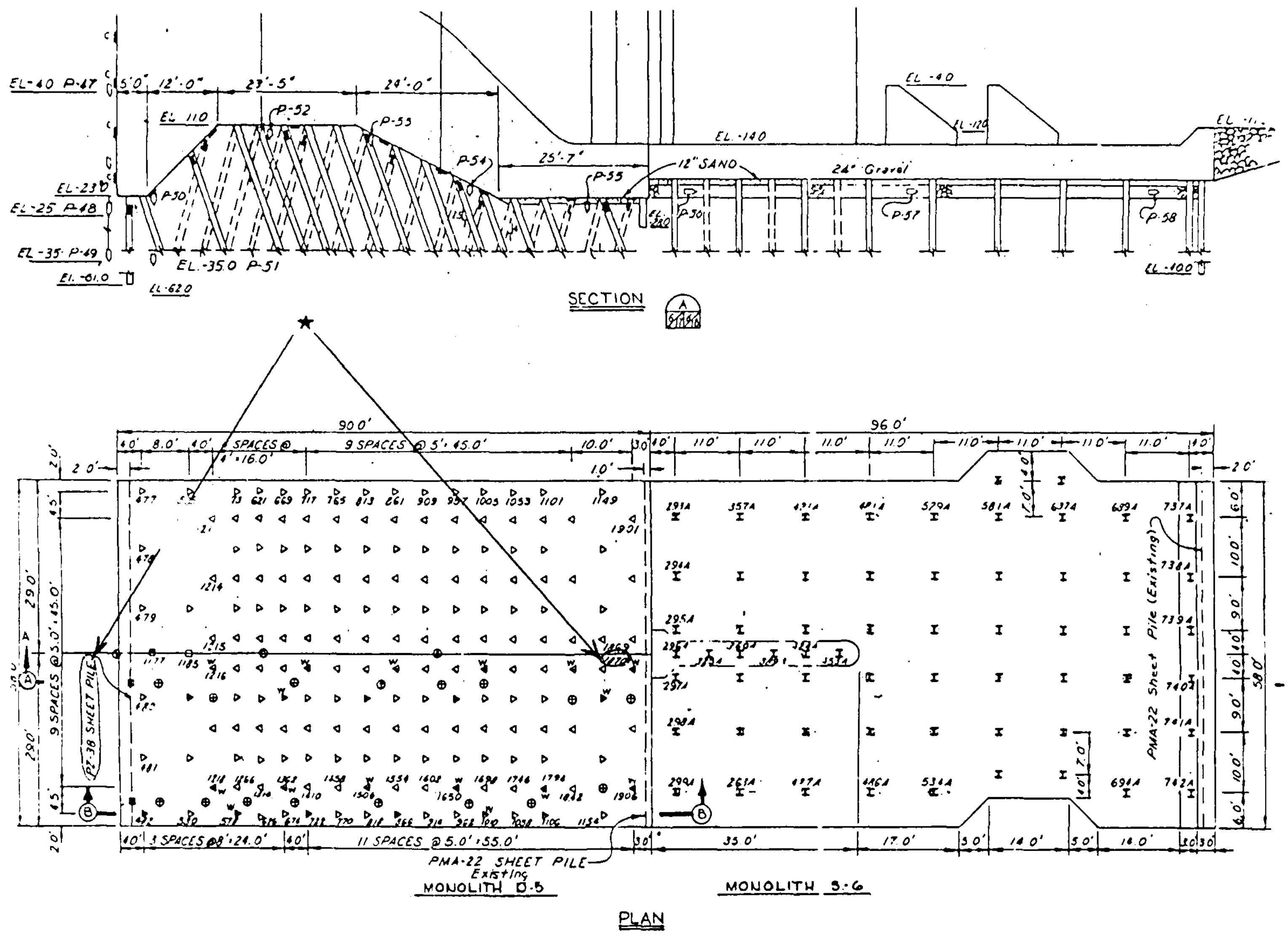


Figure 47. "H" pile and PZ-38 sheet pile locations.

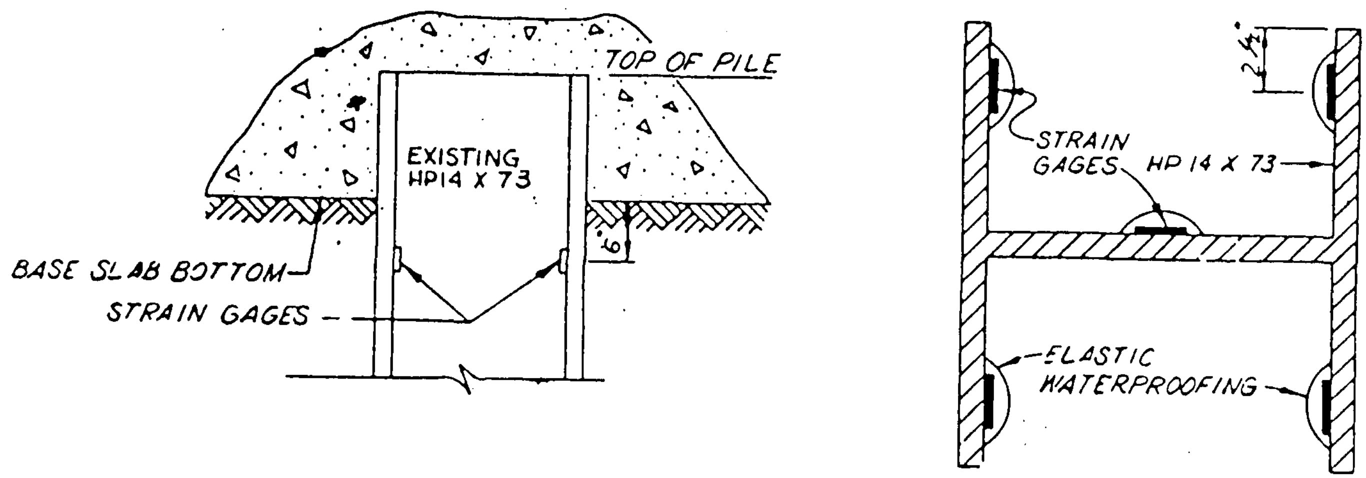


Figure 48. Gage layout for "H" pile.

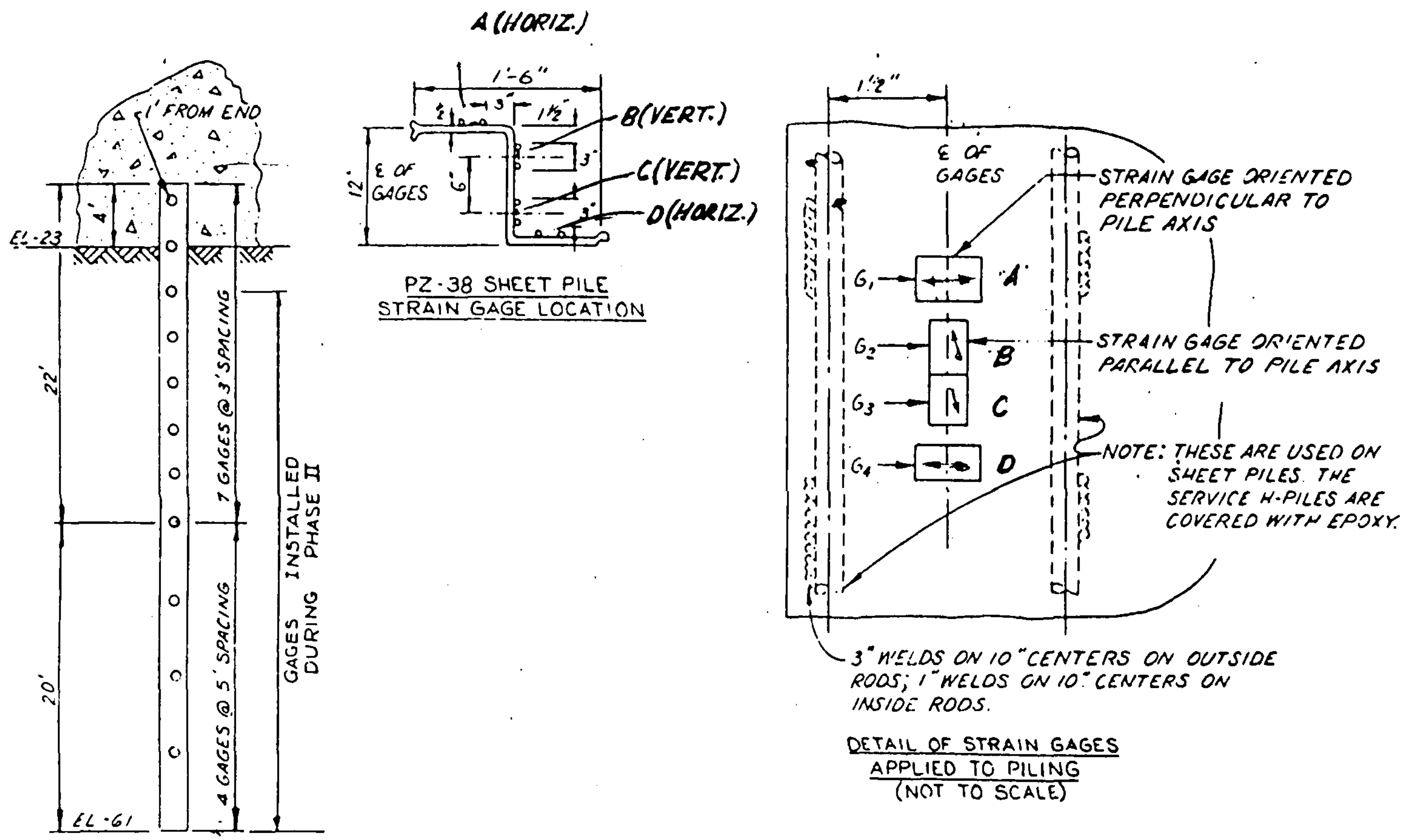


Figure 49. Gage layout for sheet pile.

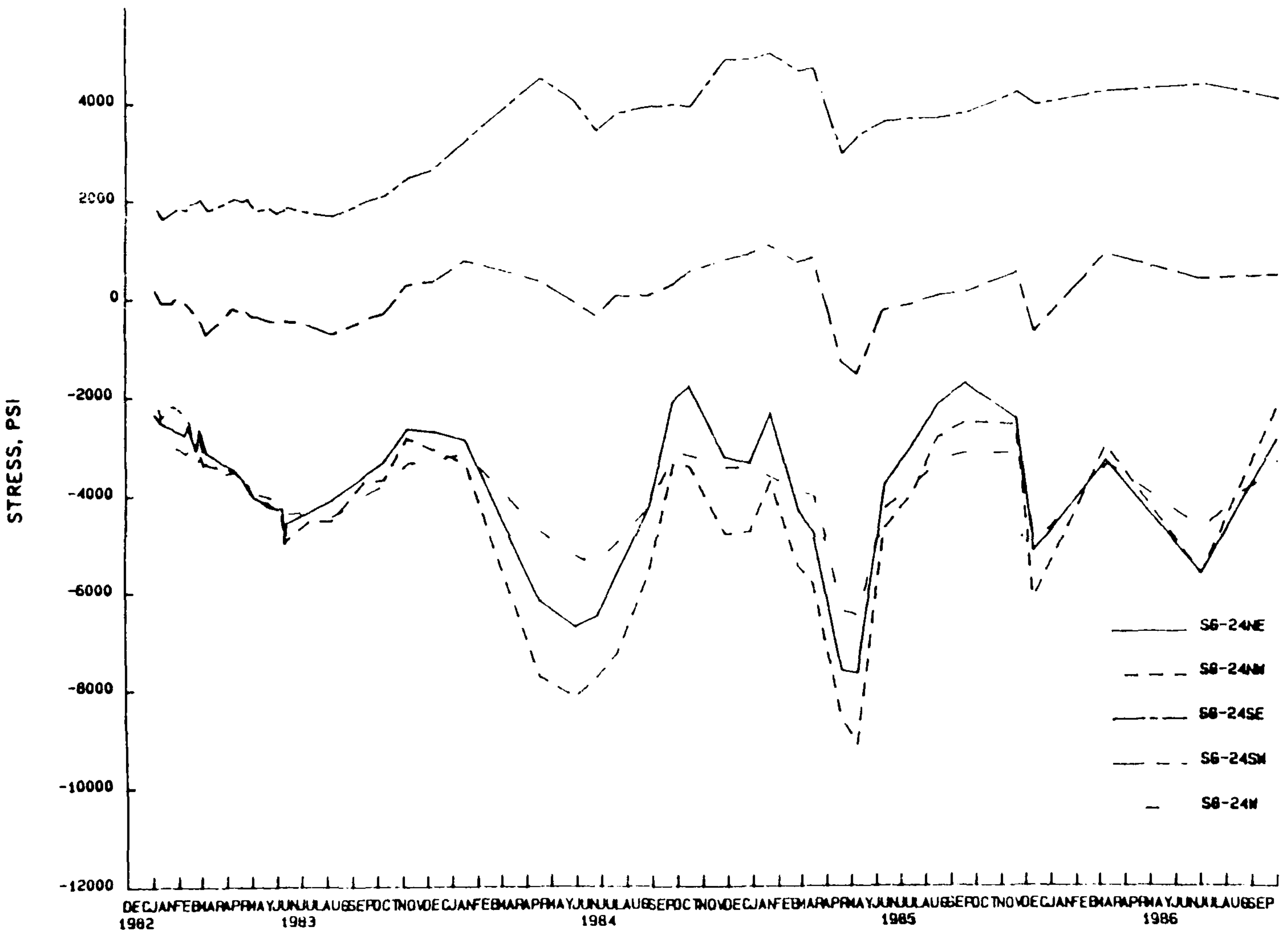


Figure 50. (B-183) Time history plots of H-pile strain gauges in dam (SG-24) 1983-1986.

transferred into the pile. During low river periods, such as the summer of 1985 and dewatering of the lock, the stresses are shown to rise and relieve the compressive load on the pile. Both these responses are expected since the river level is a primary variable load acting on the pile. Stress levels recorded during these two periods are near previously recorded levels that also correspond to high and low river levels. The maximum and minimum stress levels shown in figure 50, (5,000 psi tension and 9,150 psi compression), are well below any yield point for steel and should be within safe working stress levels. Therefore, the flood and dewatering that occurred during 1985 did not place loads on the "H" pile that were not previously experienced. A calculation is shown in figure 51 which assumes these maximum and minimum stresses occur at the same time (conservative). The pile was assumed to behave as a fully laterally braced beam/column. The stress was divided into an equivalent axial and moment load applied to the top of the pile. Analysis found that the applied stresses were only 40 percent of the allowed working stresses which indicates the pile is in no danger of failure. The recorded moment was calculated as 252 in.-kips versus an allowed value of 859 in.-kips.

90. Stress wave forms for the sheet piles are similar to the "H" pile discussed above. When the river is high the sheet pile is loaded with more compression as shown in figure 52. Conversely, the compression is relieved (axially unloaded) when the river is at low levels. All stress plots for sheet pile PZ-38 with gage designations of SG-25 through SG-46 are included in the appendix. When viewing these plots the reader should use caution to recognize that the stress scales vary and do not overlay each other. Peak recorded tension and compression values were near 20,000 psi with most stresses within ±15,000 psi. A calculation of the sheet pile maximum moment (assumed to be at the bottom of the dam, elevation -23 ft.) indicated the pile is loaded only to 50 percent of it's safe elastic working load capacity and is presented as figure 53. Shown is a recorded moment of 740 in.-kips versus 1,480 in.-kips allowed. The SG-26 records were used in this analysis since both the "B" and "C" records are complete as shown in figures B-186 and B-187 in the appendix. Recorded stress values reached a near steady state in October 1981. Peak compression of 5,000 psi and tension of 7,000 psi were recorded through September 1986 for a total range of 12,000 psi. The corresponding gage data (SG-37) for the other sheet pile reached similar steady states of stress in

Given: HP 14 x 73 (AISC Manual)

Area = 21.4 in^2 $S_x x$ = 107 in^3 $S_y y$ = 35.8 in^3

$r_x = 5.84$ $r_y = 3.49$

Assume: Rolled section and F_y = 36 ksi

$F_b = .66\ F_y$ allowed stress

F_b = .66(36 ksi) = 24 ksi

Fully laterally supported kL/r < 1

F_a = 21.5 ksi

Moment allowed

$M_a x = F_b S_x x$ = 24 ksi(107 in^3) = 2,568 $in^{\cdot k}$ ⟵ strong axis

$M_a y = F_b S_y y$ = 24 ksi (35.8 in^3) = $\boxed{859\ in^{\cdot k} \longleftarrow \text{weak axis}}$

Recorded load (just below dam):

Maximum 5,000 psi (tension) -9,150 psi (compression)

Find equivalent axial and moment, assume elastic response and tension equals compression for the moment

Range = 5,000 + 9,150 = 14,150 psi

1/2 of range is compression and 1/2 is tension gives:

7,050 psi (tension) 7,050 psi (compression) = f_b

Shift these by an axial load of -2,050 to get data values = f_a

-2,050 (compression) - axial

5,000 (tension) - moment
-9,150 (compression) - moment } agrees with data

Thus,

Applied axial stress = f_a = -2,050 psi = axial load

Axial load = $f_a(A)$ = (2,050 psi)(21.4 in^2)
= $\boxed{43,870\ \text{lbs} \downarrow\ = \text{load}}$

Applied moment = $M_a = Sf_b$ = (35.8 in^3)(7,050 psi)
= $\boxed{252\ \text{in} = M_a\text{pplied}}$

Equivalent loads from recorded data

43,870 lbs

Moment = 252 $in^{\cdot k}$

(Continued)

Figure 51. Analysis of "H" pile.

Allowable loads (Use weak axis)

$$f_a = \frac{P}{A} = 2,050 \text{ psi} \qquad f_b = \frac{M}{S} = 7,050 \text{ psi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{20.5}{21.5} + \frac{7.05}{24} = 0.39 < 1 \quad \text{Steel is not overloaded}$$

Summary

"H" pile only loaded 40 percent; steel elastic allowable stress has been used assuming bending about weakest axis.

Figure 51. Concluded.

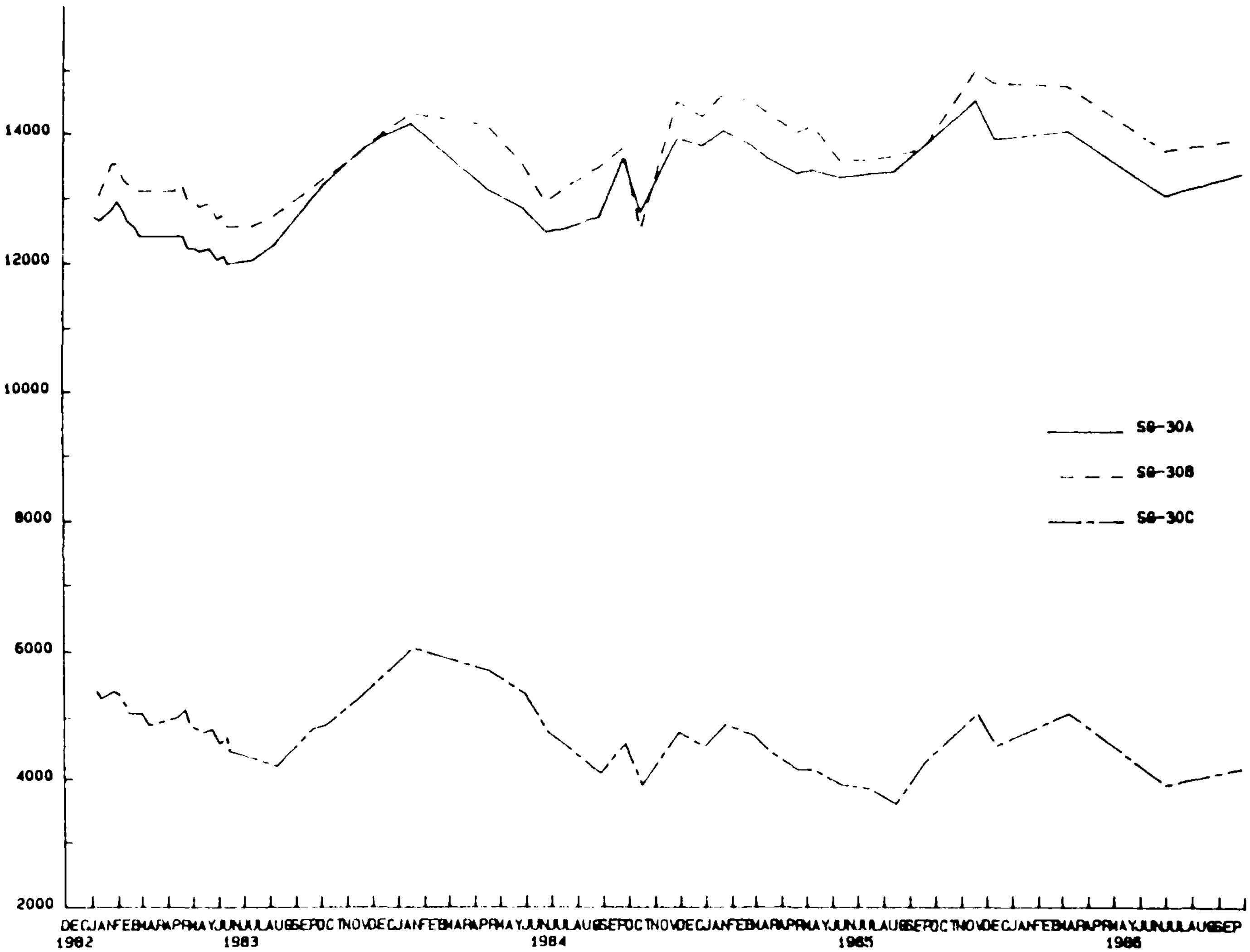


Figure 52. (B-195) Time history plots of sheet pile strain gauges in dam (SG-30) 1983-1986.

Data: Use assumed maximum moment location just below dam (SG-26)

Recorded maximum 7,000 psi tension

5,000 psi compression

Assume elastic response and moment has equal tension and compression, range is 7,000 + 5,000 = 12,000 psi

1/2 of range is compression and 1/2 is tension and gives:

6,000 psi (tension) 6,000 psi (compression)

Shift these by 1,000 psi tension to get data values

Axial uplift of 1,000 psi

$\left.\begin{array}{l}\text{7,000 psi tension}\\ \text{5,000 psi compression}\end{array}\right\}$ agrees with data

$\boxed{\text{Axial stress is +1,000 psi}}$

Due to strain gages being 3 inches from neutral axis and the flanges being 6 inches from the neutral axis, the stress values will be doubled to work with the flanges.

Thus assume:

$\boxed{\left.\begin{array}{l}\text{12,000 psi tension stress in flange}\\ \text{12,000 psi compression stress in flange}\end{array}\right\} \text{moment}}$

Find section properties (Refer to Figure 49 for details)

Area = 1/2 in(18 in + 12 in) = 1/2(30) = 15 in^2

Assume bending can be only in direction of 12" depth, perpendicular to the sheet pile wall.

$$C = 6 \text{ in}$$

$$I = \frac{9 \text{ in}(12^3 - 11^3)}{12} + \frac{1/2(12)^3}{12} = 370 \text{ in}^4$$

$$S = \frac{I}{C} = \frac{370 \text{ in}^4}{6 \text{ in}} = 61.7 \text{ in}^3$$

$$\boxed{M_{allowed} = M_y = SF_b = (61.7)(24) = 1{,}480 \text{ in}^{-k}}$$

Find applied loads (axial and moment)

Axial: $P = f_aA = 1{,}000 \text{ psi}(15 \text{ in}^2) = \boxed{15{,}000 \text{ lbs} \uparrow}$

Moment: $M_a = SF_b = (61.7 \text{ in}^3)(12{,}000 \text{ psi}) = \boxed{740 \text{ in}^{-k}}$

(Continued)

Figure 53. Analysis of Sheet Pile

Allowable

Assume no buckling, fully lateral support, $f_y = 36$ ksi

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{1}{21.5} + \frac{12}{24} = 0.55 < 1$$

Summary

Sheet pile is only loaded to 50 percent of allowable working load.

Figure 53. Concluded.

the fall of 1981 but does not provide complete records for the "C" gage; however, they do add creditability to the measurements. A moment diagram and maximum axial stress, versus distance below the dam are shown in figure 54. The "B" and "C" gage data were used assuming elastic response and a resulting ideal moment and axial load for a laterally and axial loaded beam. These stress levels are not near any assumed yield stress for steel and thus indicate the loads on the sheet pile are within working stress levels. Summary of the calculations is shown in table 7.

91. Several of the sheet pile data plots have large changes in stress very early (1980-1981). This response is assumed due to pile driving, earthwork, and resulting residual stresses and is not considered as normal operation variations and thus are not analyzed here. Typically, the "B" and "C" data has a range less than the 12,000 psi used in the calculations above. An exception is gage SG-34 which is suspect to error since the "B" data plot is incomplete and does not track from figure A-20 to figure A-23 in the appendix.

92. Readings from the instrumentation data indicate that the single "H" pile and the two sheet piles are performing as designed. This is not sufficient data to state that all "H" piles and sheet piles are performing within design limits; however, since no unusual displacements or cracking is taking place on the concrete dam section these data give further verification that this section of the dam is performing as designed.

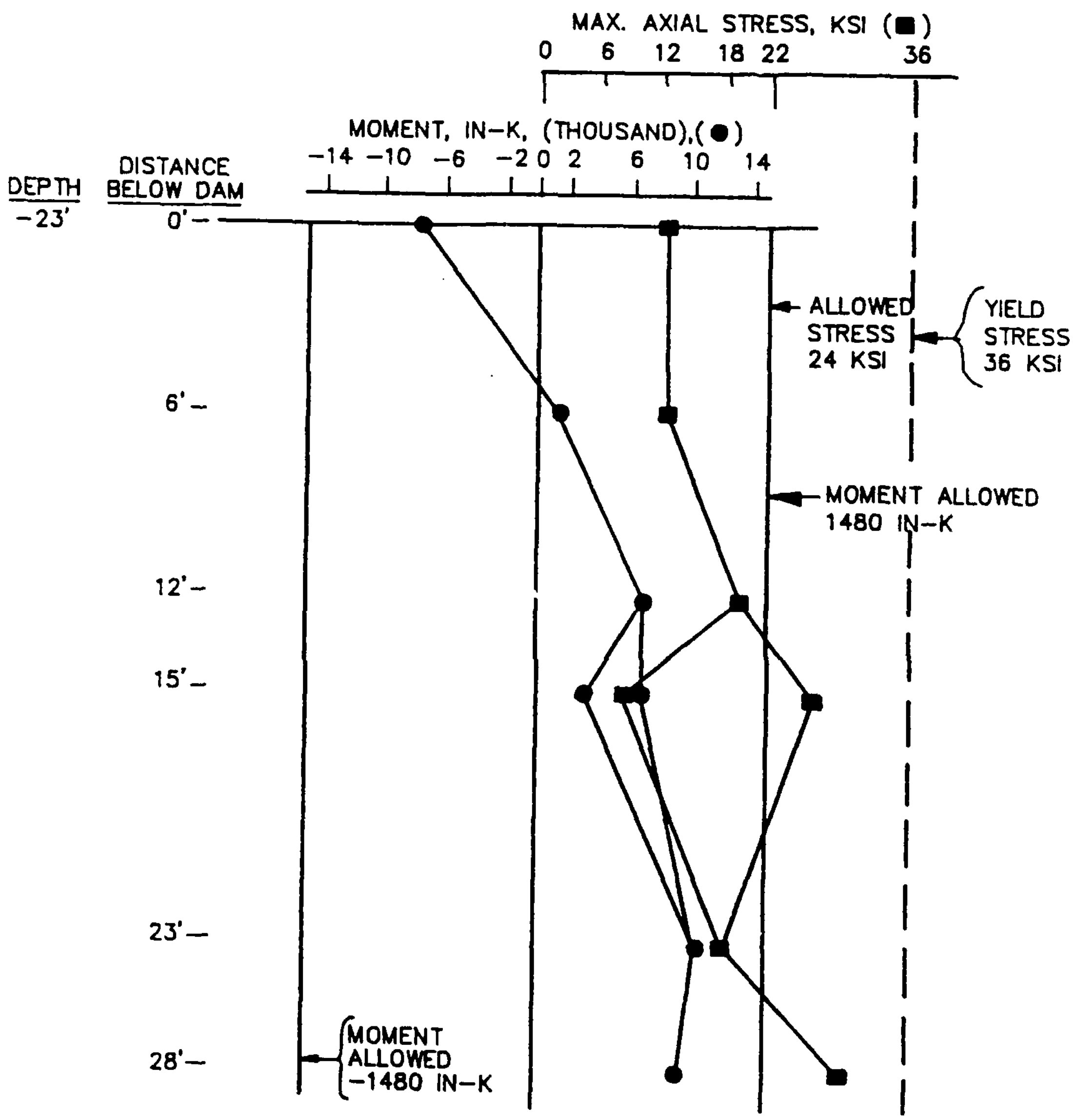


Figure 54. Sheet pile moment and axial distributions.

Table 7. Sheet Pile Analysis

Depth ft	Gage	Recorded ksi B	Recorded ksi C	Range ksi	Comp/tension 1/2 range (ksi)		Axial shift	Moment sign	Moment Flange Stress ksi		Max Total Stress ksi	Applied Moment, in-k
0	SG-26	-5	+7	+12	-6	6	+1,000	+	-12	+12	+13	-740
6	SG-28	+12.2	+9.5	2.7	1.3	-1.3	+10,900		2.6	-2.6	+13.5	+160
12	SG-30	+15	+3.8	11.2	5.6	5.6	+9,400		11.2	-11.2	+20.6	+691
15	SG-31	+22	+11	11	5.5	5.5	+16,500		11	-11	+27.5*	+679
15*	SG-42	0	5.8	5.8	2.9	2.9	+2,900		5.8	-5.8	+8.7	+358
23*	SG-44	+10	7.5	17.5	8.8	8.8	+1,200		17.6	-17.6	+18.8	+1,086
28	SG-34	+22	+6	16	8	-8	+14,000		16	-16	+30*	+987

* These two total stresses are over allowed elastic working stress; but, have not reached yield of 36 ksi and thus should still behave elastically.

PART X: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Instrumentation

93. In general, most of the instrumentation seems to be functioning well. The number and types of devices installed were adequate except for the inclinometers, settlement plates and heave points.

94. Inclinometer readings were unreliable and not used in determining deformation of the lock walls. There was no calibration for the twist to provide an accurate measurement. The direct measurement of the reference bolts was the only acceptable data.

95. The settlement analysis could have been more accurate if the settlement plate elevations were properly recorded initially and read as scheduled. The settlement plates were difficult to locate at the bottom of the lock floor and were not read.

96. The soil pressure meters were very consistent and believed to be accurate.

97. Field personnel also expressed some reservation in the methods of reading the surface monuments. Reading methods varied from one party to another; however, there was no significant difference in the overall surface monument data.

Observation and Analysis

98. In general, observed settlements were less than predicted settlements. The settlements for the construction condition were higher than the predicted values. The reason may be that the actual rebound is less than that assumed. This assumption cannot be verified since there are no records for heave points. Another reason for the differences is probably due to the variance in the actual condition from the assumed case.

99. The observed base pressures along the base slab were considerably higher than the predicted values. In December 1981, the soil pressure meters along the wing wall experienced a drastic change in the pressures during construction of the backfill due to the settlement of the soil below it. The observed high base pressure may reflect the load transfer from the wing to the base slab. Another explanation for the high base pressure could be the result

of silt accumulation at the bottom of the lock. The shape of the pressure distribution curves also differed from that of the trapezoidal shape assumed in design. The actual curves have peaks occurring along the slab which are constant with the deformation of the base slab (figure 36). The observed lateral earth pressures above the base slab were in fair agreement with the predicted values, especially along the upper half of the wall.

100. The moment analyses indicate that, within the base slab, CUFRAM provides reasonable agreement with moments and deflections obtained from instrumentation. Deflections based on moment calculations and on strain data show good agreement; however, no valid deflection measurements are available for comparison. Moments calculated using uniform base pressure distribution are approximately equal to moments calculated based on strain data. Moments calculated using measured base pressure distributions are, however, much larger than those obtained using the uniform base pressure. Moments in the wall section and moments in the culverts do not show good agreement with moments obtained using strain gage data. In the case of the wall, this may be due to incorrect measurement of some of the strains. In the case of the culverts, the relatively short members make it unlikely that the analyses will provide reasonable results.

Recommendations

Recommendation regarding Lock and Dam No. 1

101. Inclinometers should be checked for twist and read once yearly during the low-water season. Other instrumentation should be monitored as scheduled and for unusual loading conditions.

Recommendations regarding similar structures

102. The importance of continuity of data acquisition is emphasized and instrumentation personnel should receive adequate training and instruction.

LITERATURE CITED

1. Andersen, J. and Vanadit-Ellis, W.,"Instrumentation Data Summary, Lock and Dam No.1, Red River Waterway, LA," November 1987, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

2. Sherman, W.C. and Trahan, C.C.,"Analysis of Data from Instrumentation Program, Port Allen Lock," September 1968, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

3. "Lock and Dam No.1, Detail Design Phase 2, Soils Data and Foundation Design; Red River Waterway, LA, Tex., Ark., and Okla. Mississippi River to Shreveport, LA, " Design Memorandum No. 9, July 1977, U.S. Army Engineer, New Orleans District, New Orleans, LA.

4. "Foundation Report, Red River Waterway, LA, Lock and Dam No. 1," September 1984, U.S. Army Engineer, New Orleans District, New Orleans, LA.

5. Dunnicliff, J., Geotechnical Instrumentation for Monitoring Field Performance, John Wiley & Sons, New York, 1988.

6. "Red River Waterway, LA, Tex., Ark., And Okla. Mississippi River to Shreveport, LA; Detail Design Memorandum No. 11, Masonry, Embedded Metals, Gates and Operating Equipment, Lock and Dam No. 1, Volume I through IV, U.S. Army Engineer, New Orleans District, New Orleans, LA.

7. Dawkins, W.P., User's Guide "Computer Program for Two Dimensional Analysis of U-Frame Structures (CUFRAM)", Instrumentation Report ITL-87-1.

APPENDIX A

RECORDED PILE STRESS DATA

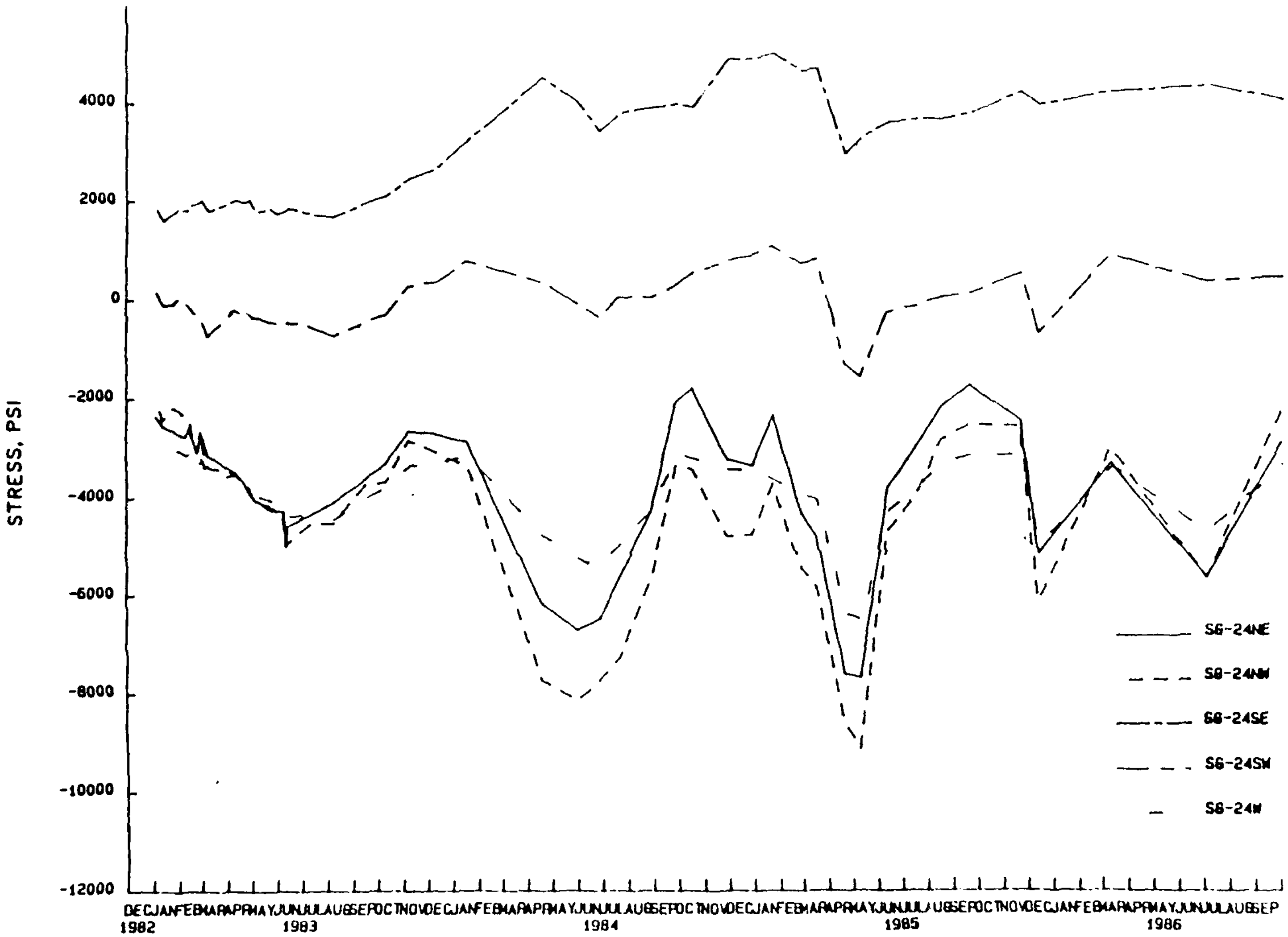


Figure A-1. (B-183) Time history plots of H-pile strain gauges in dam (SG-24) 1983-1986.

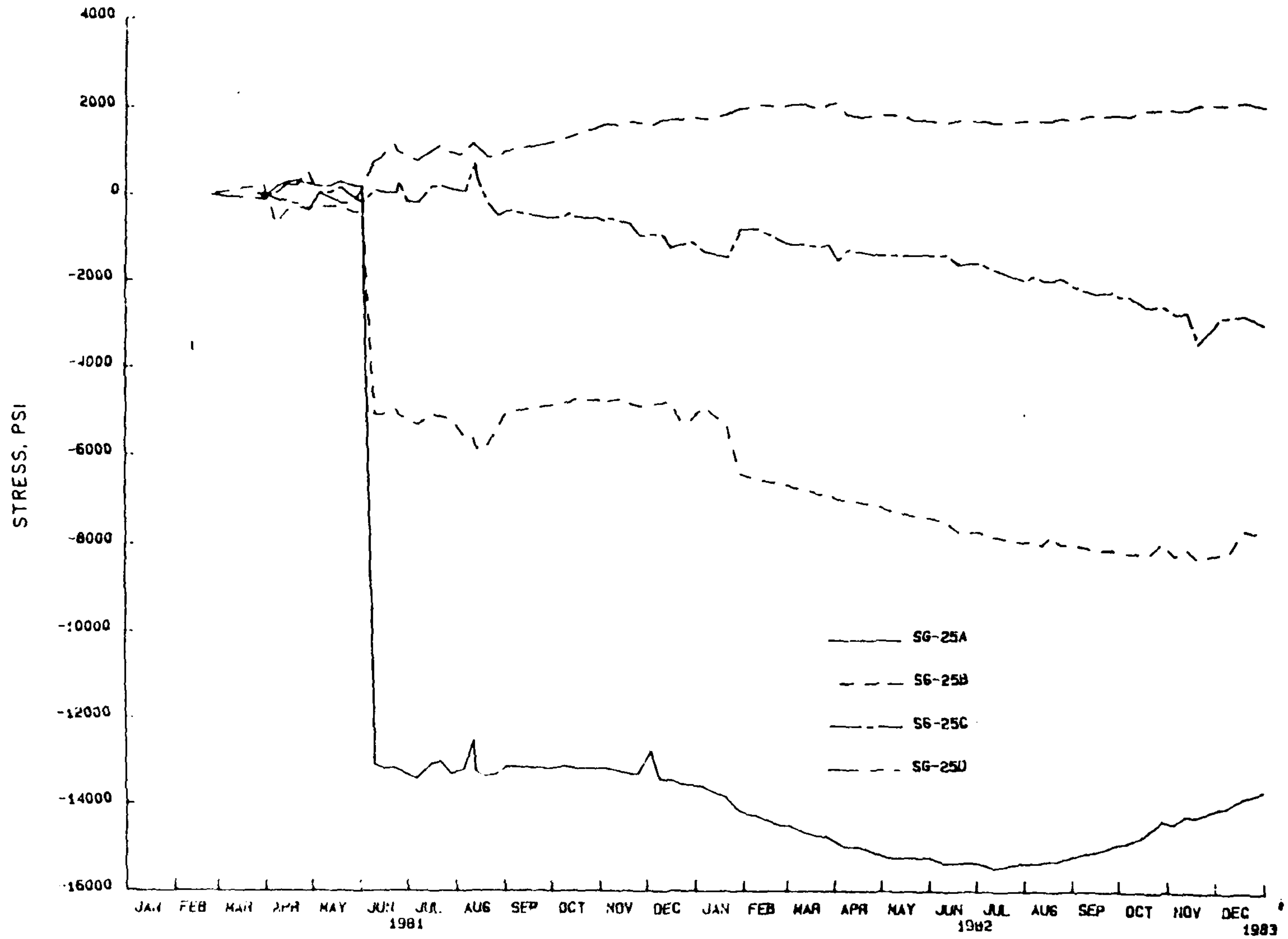


Figure A-2. (B-184) Time history plots of sheet pile strain gauges in dam (SG-25) 1981-1982.

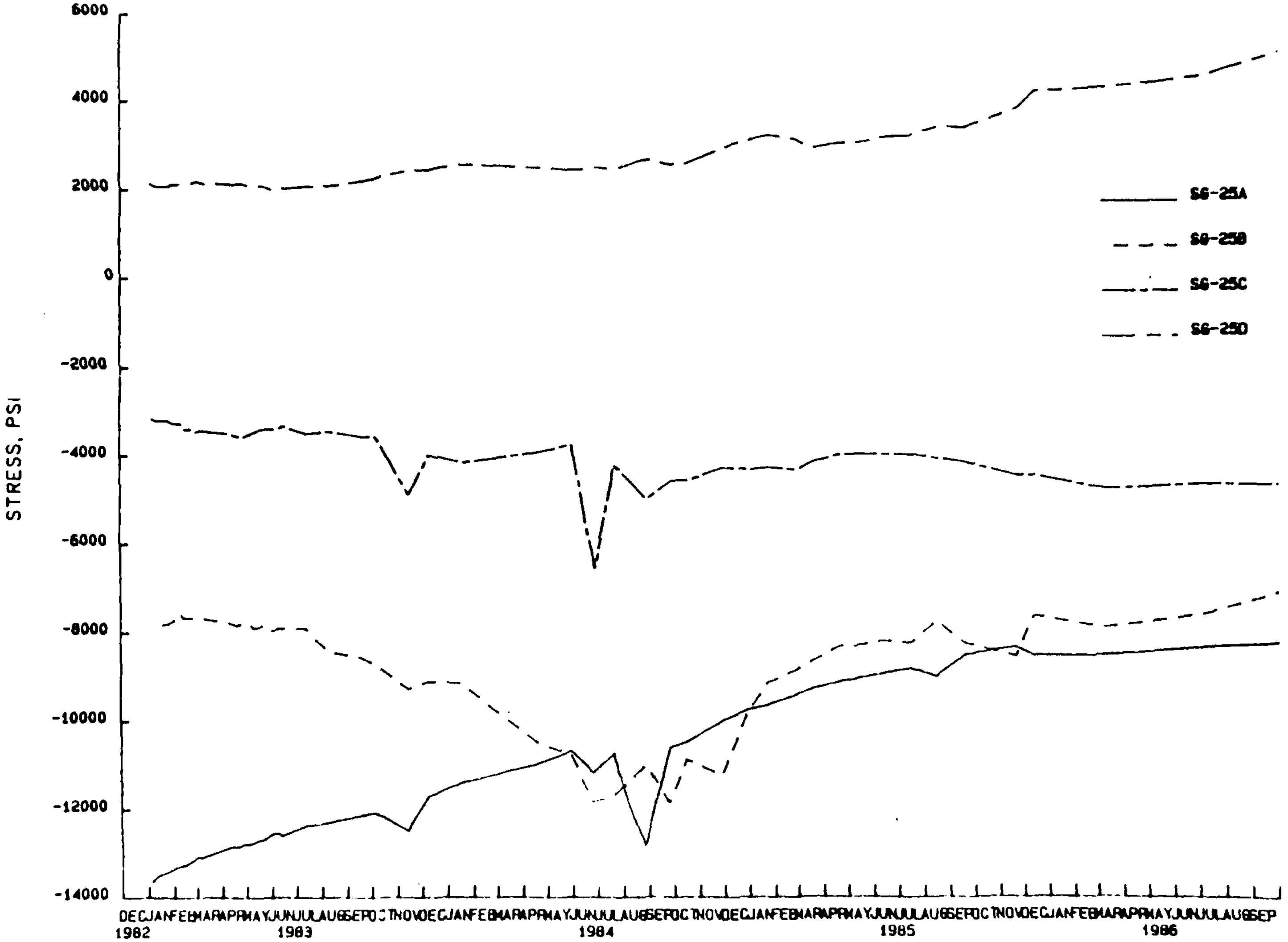


Figure A-3. (B-185) Time history plots of sheet pile strain gauges in dam (SG-25) 1983-1986.

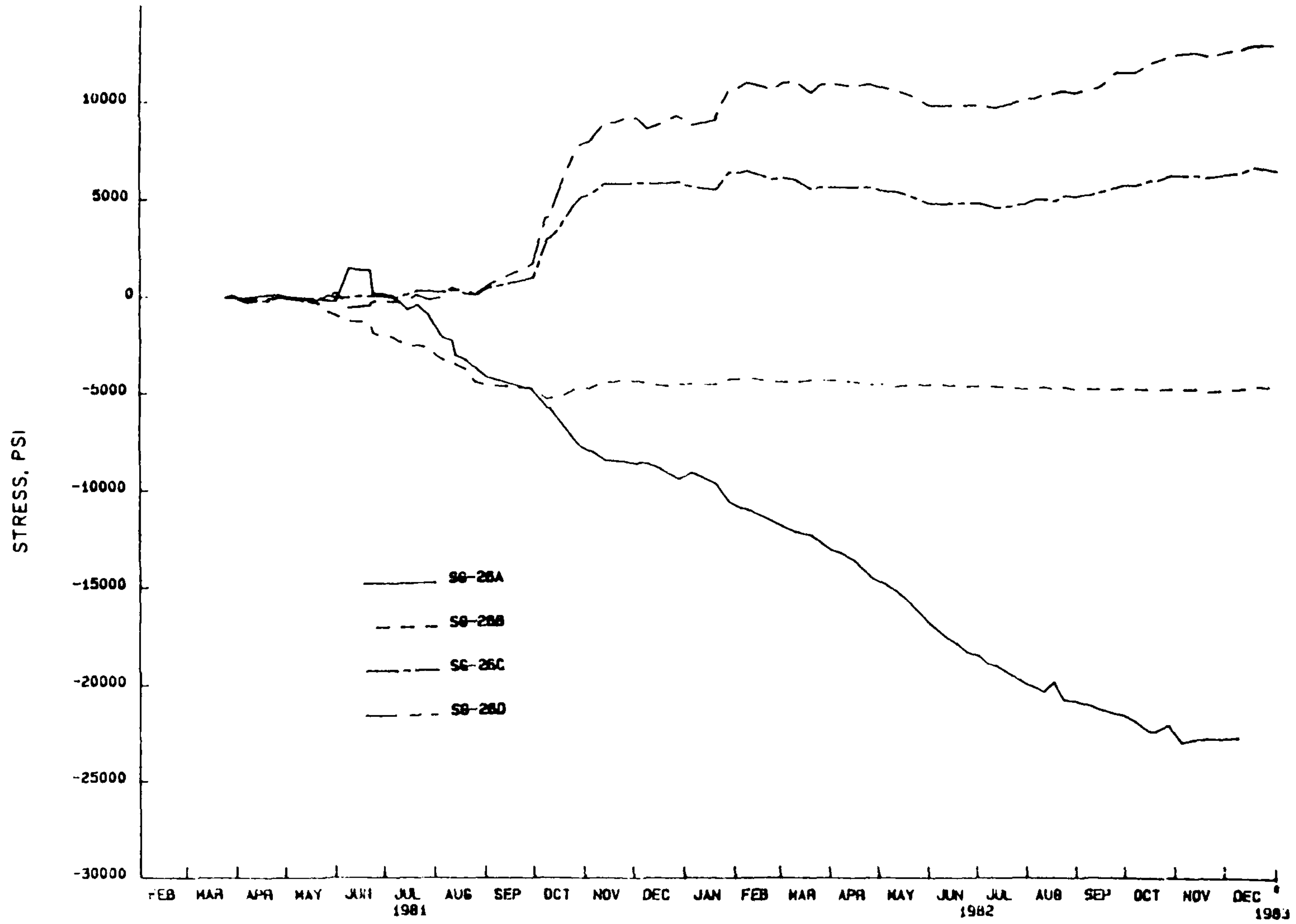


Figure A-4. (B-186) Time history plots of sheet pile strain gauges in dam (SG-26) 1981-1982.

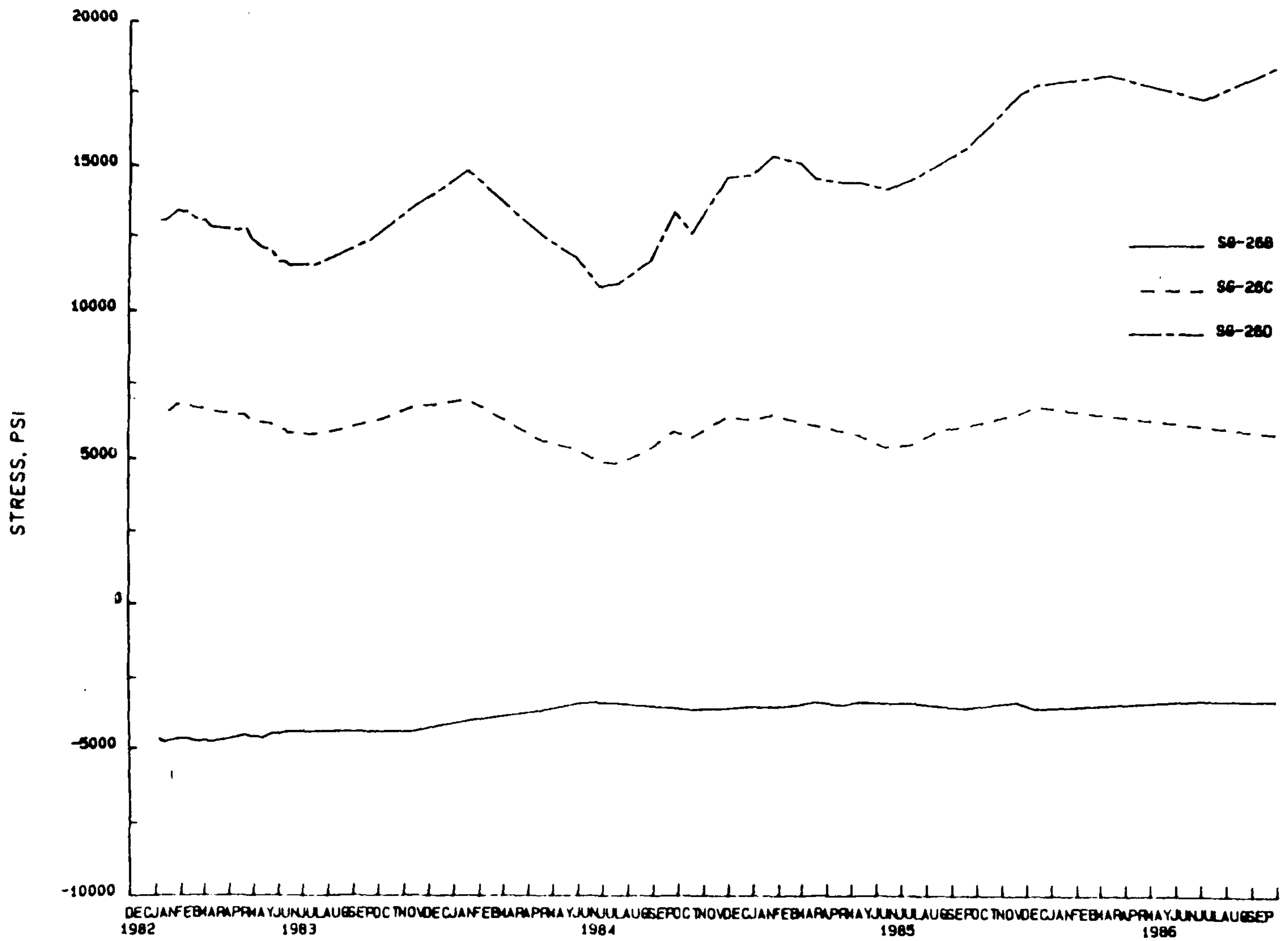


Figure A-5. (B-187) Time history plots of sheet pile strain gauges in dam (SG-26) 1983-1986.

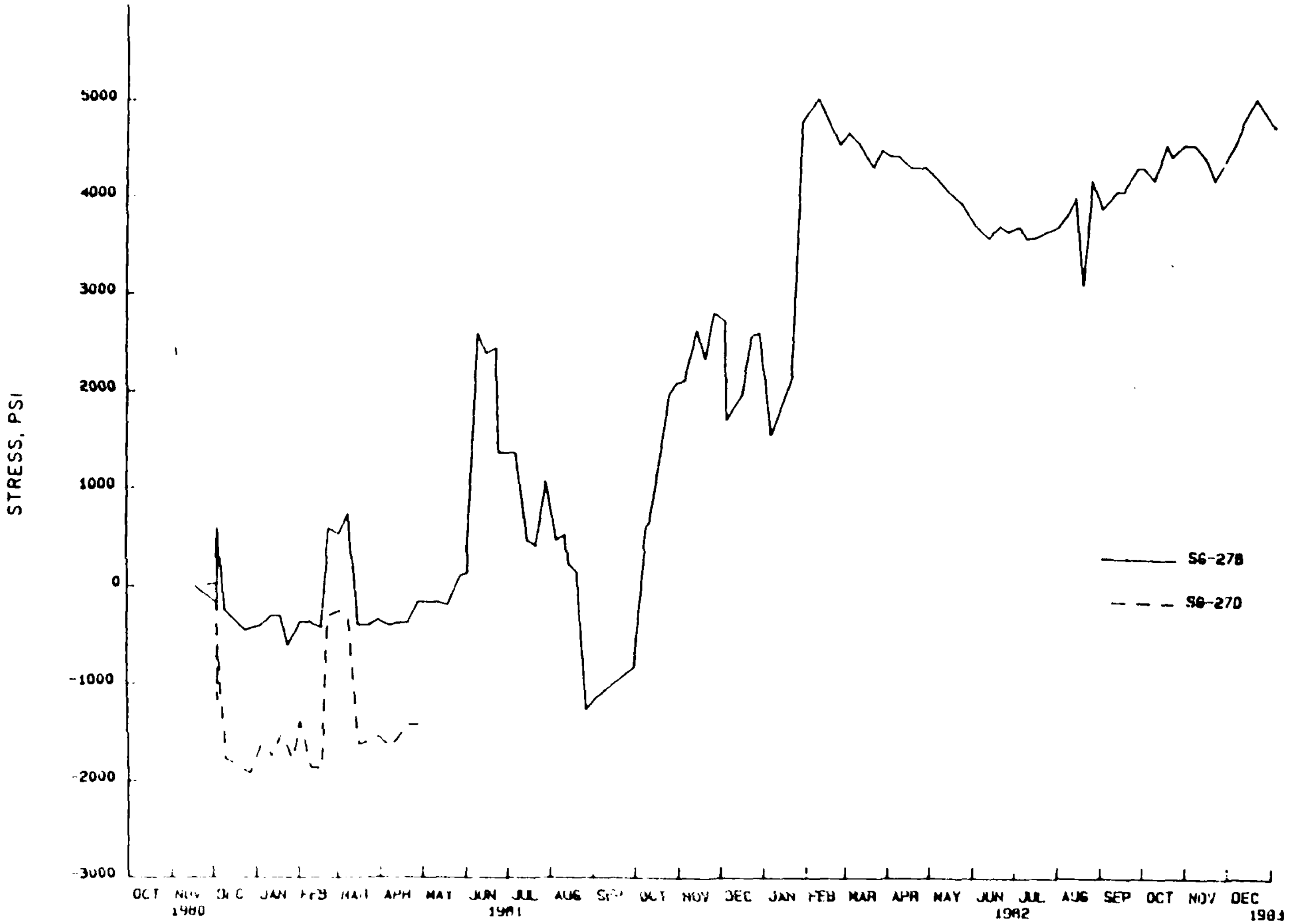


Figure A-6. (B-188) Time history plots of sheet pile strain gauges in dam (SG-27) 1981-1982.

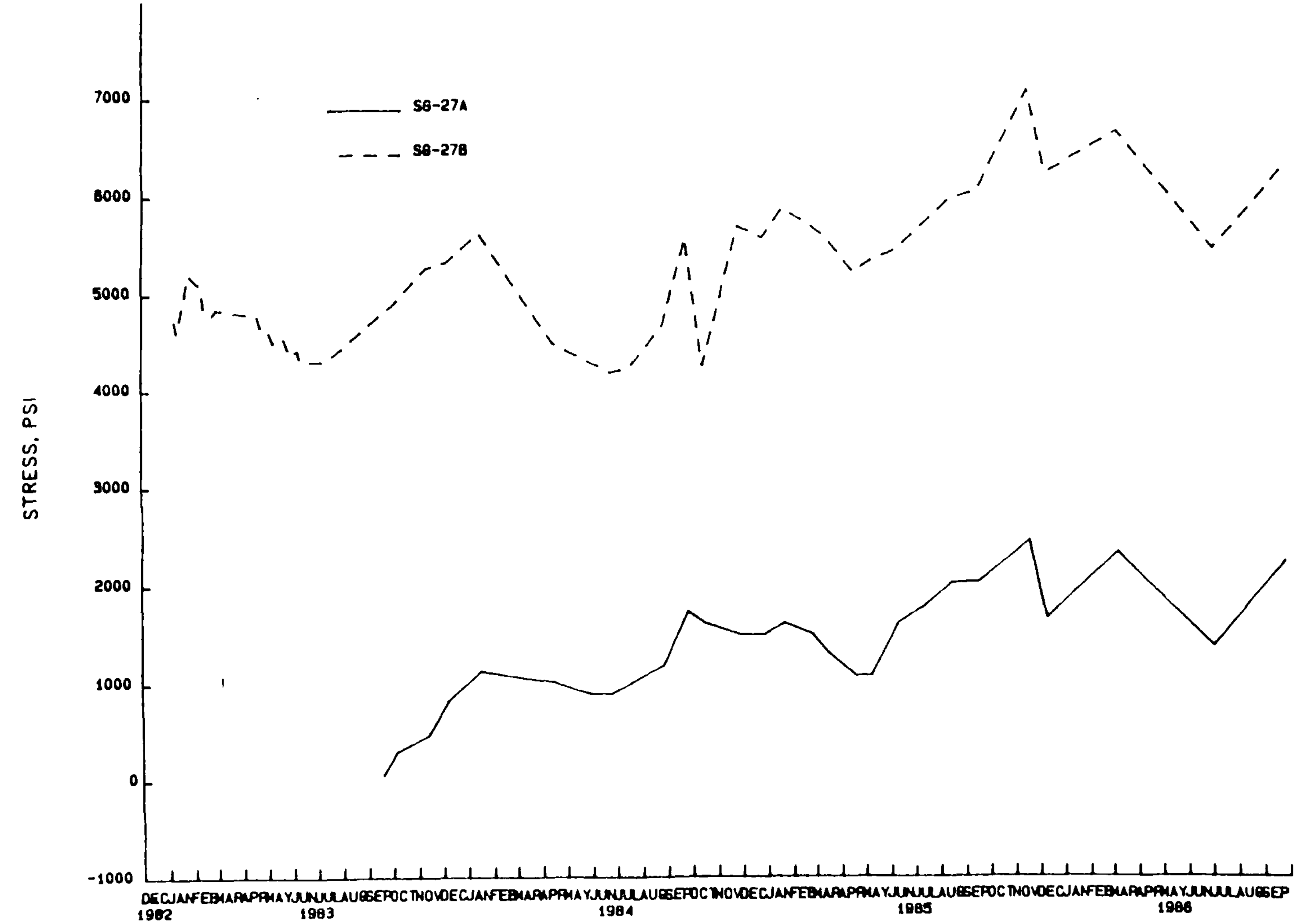


Figure A-7. (B-189) Time history plots of sheet pile strain gauges in dam (SG-27) 1983-1986.

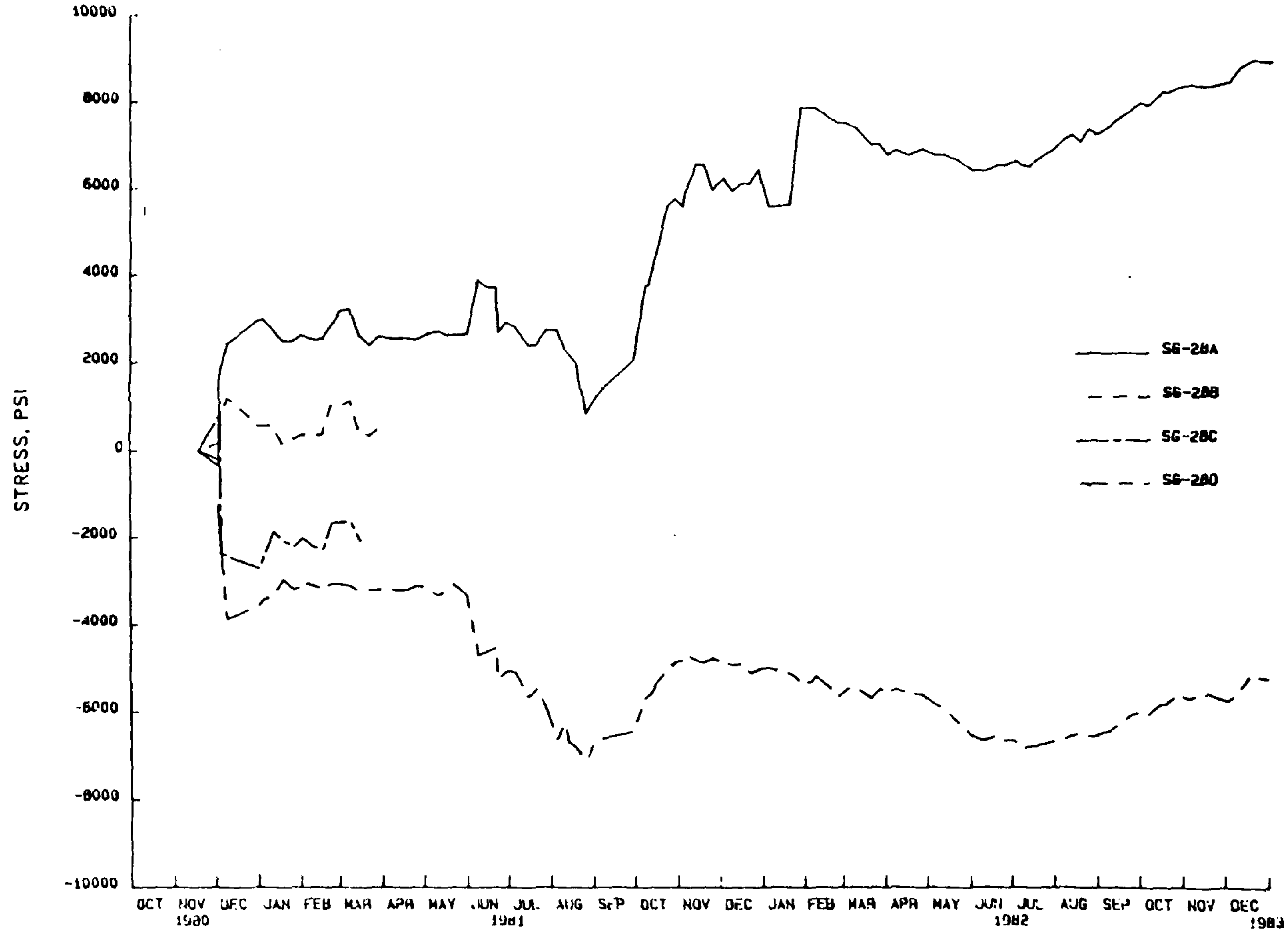


Figure A-8. (B-190) Time history plots of sheet pile strain gauges in dam (SG-28) 1981-1982.

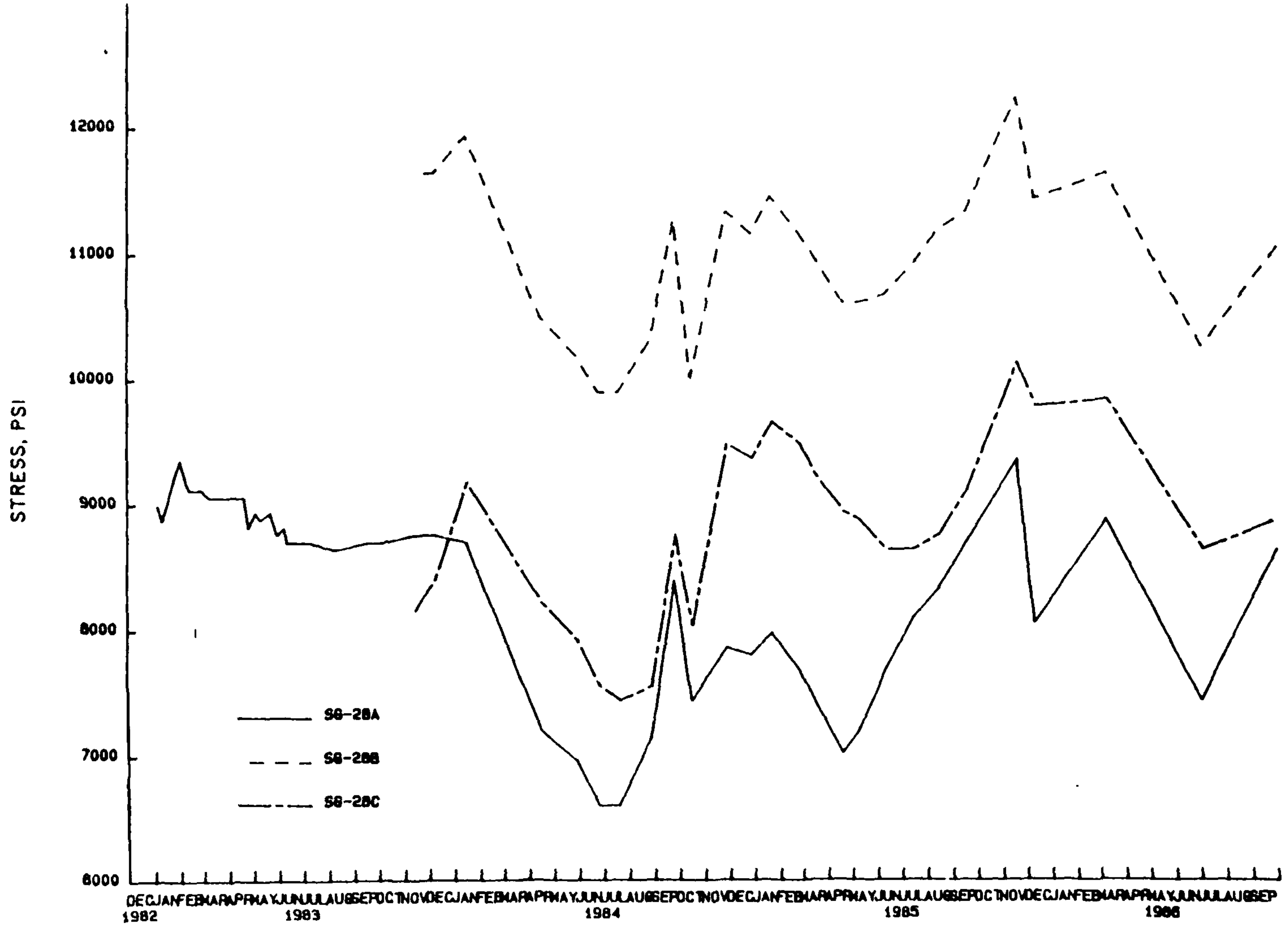


Figure A-9. (B-191) Time history plots of sheet pile strain gauges in dam (SG-28) 1983-1986.

Figure A-10. (B-192) Time history plots of sheet pile strain gauges in dam (SG-29) 1981-1982.

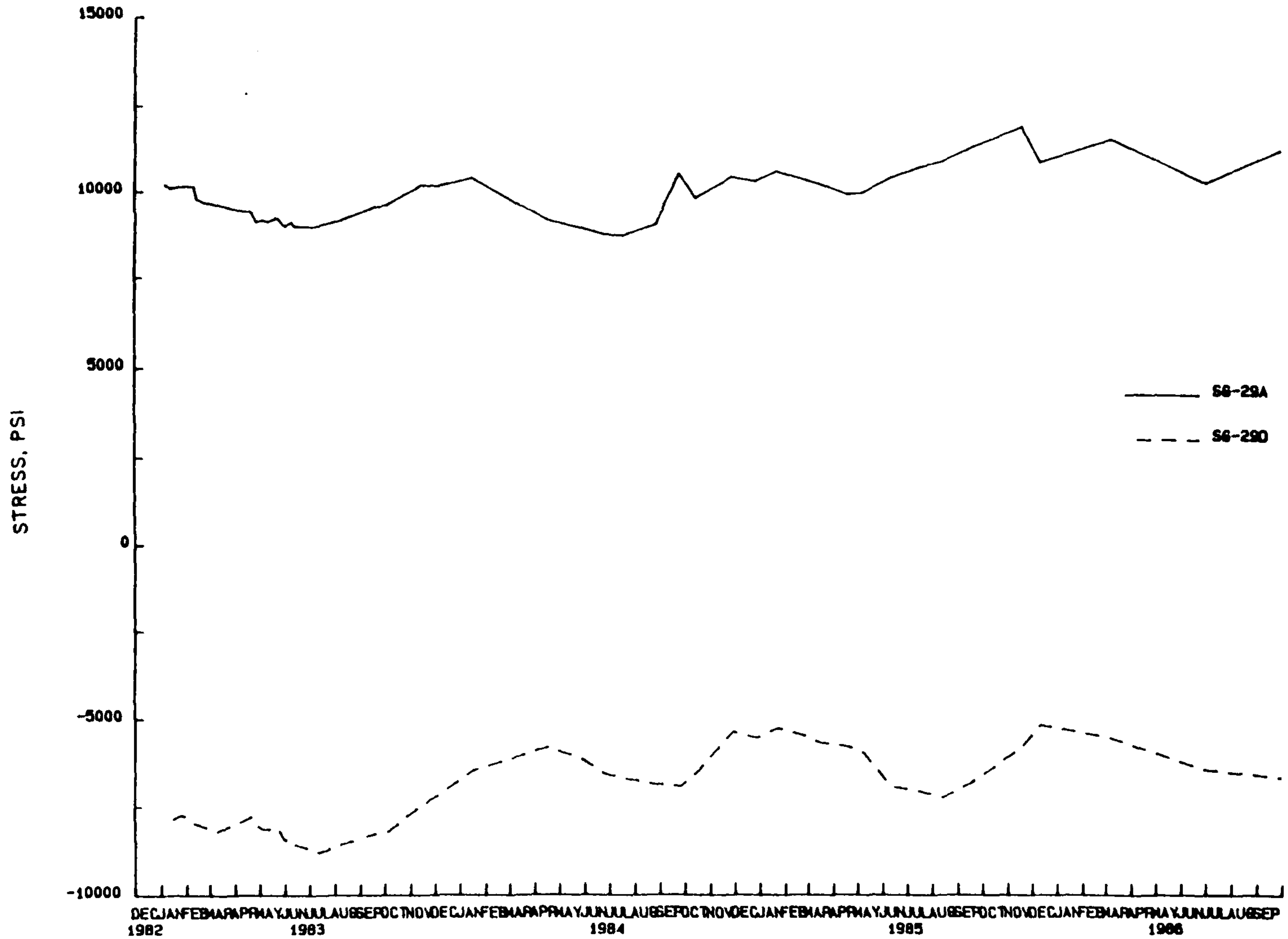


Figure A-11. (B-193) Time history plots of sheet pile strain gauges in dam (SG-29) 1983-1986.

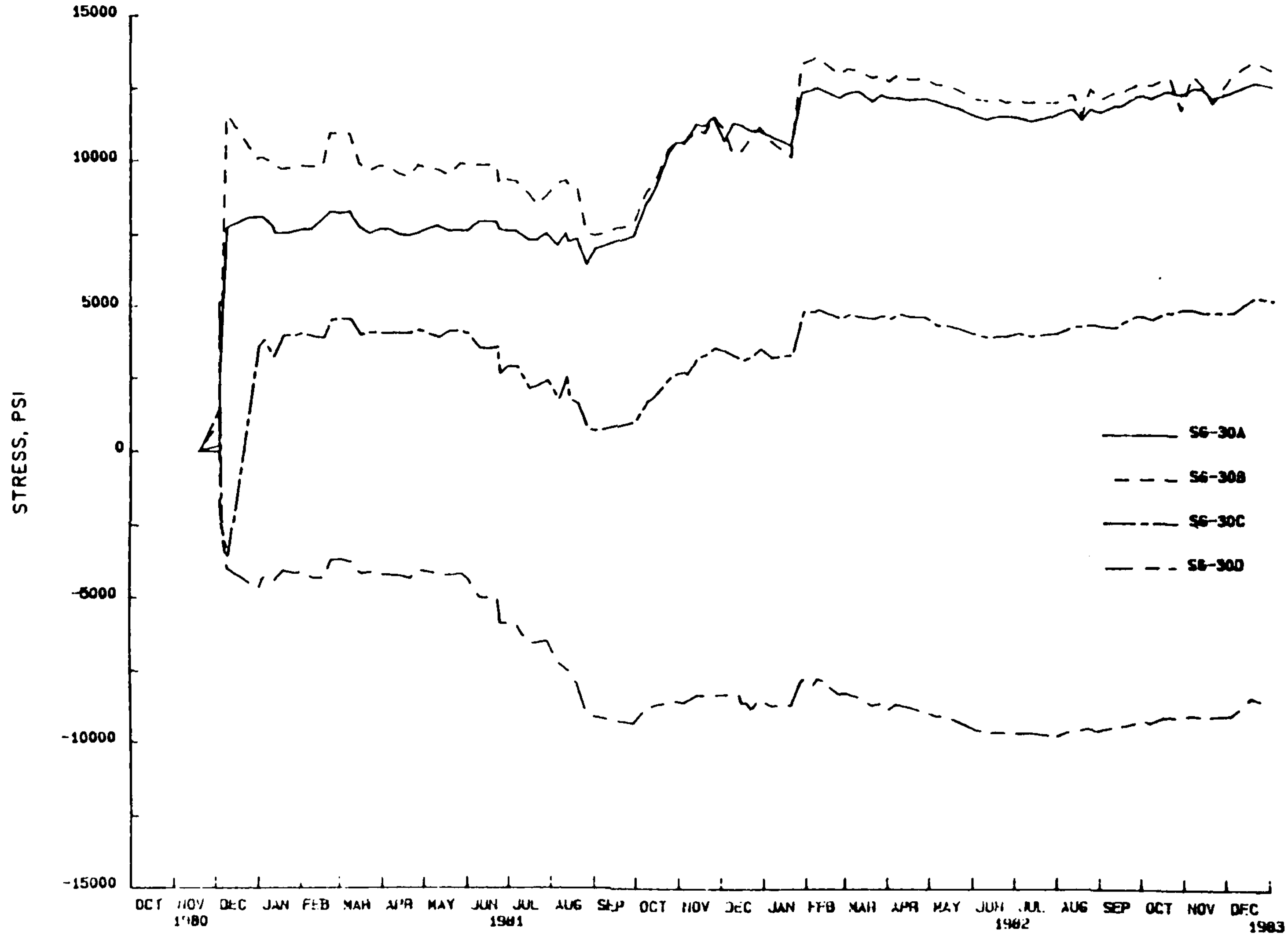


Figure A-12. (B-194) Time history plots of sheet pile strain gauges in dam (SG-30) 1981-1982.

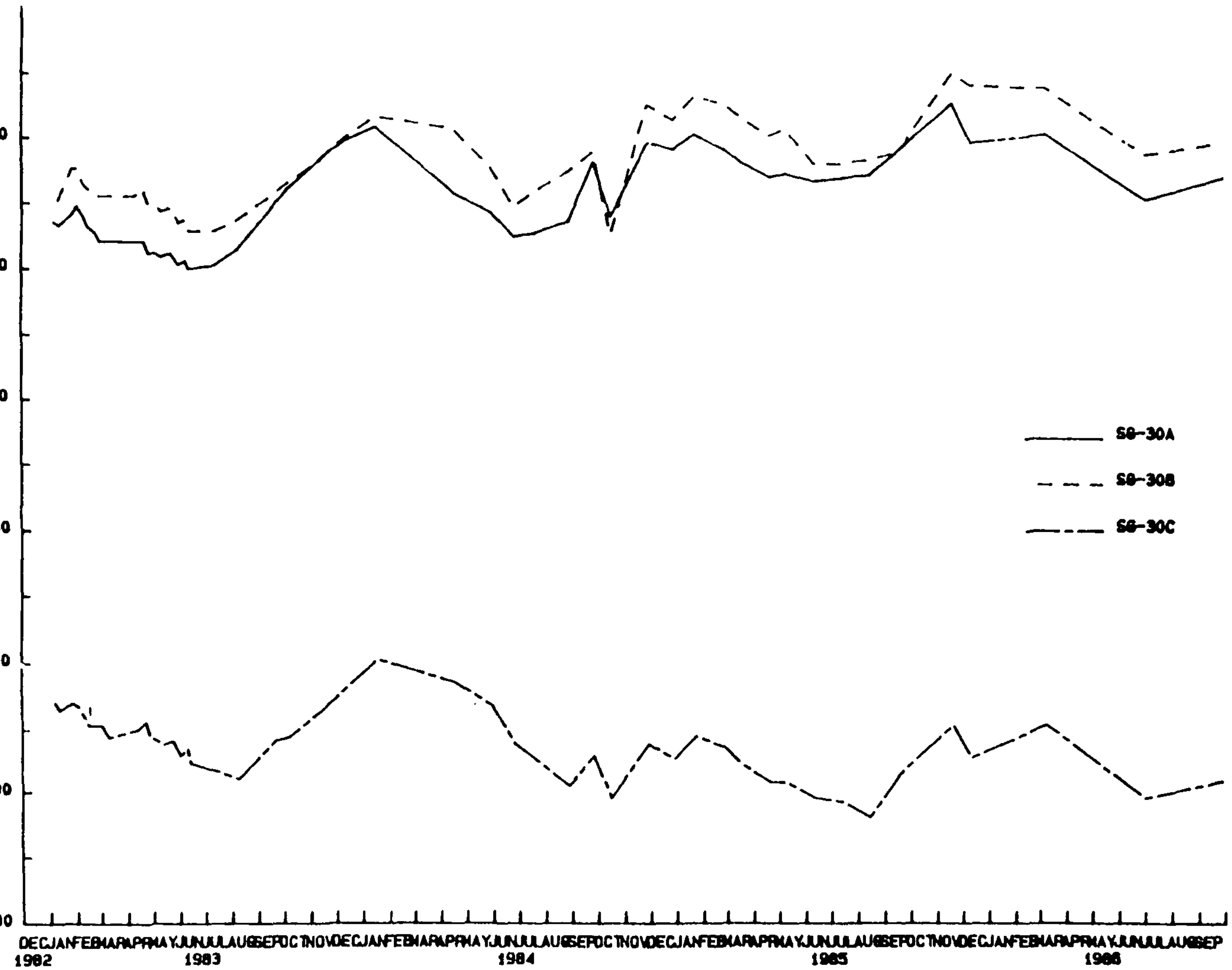


Figure A-13. (B-195) Time history plots of sheet pile strain gauges in dam (SG-30) 1983-1986.

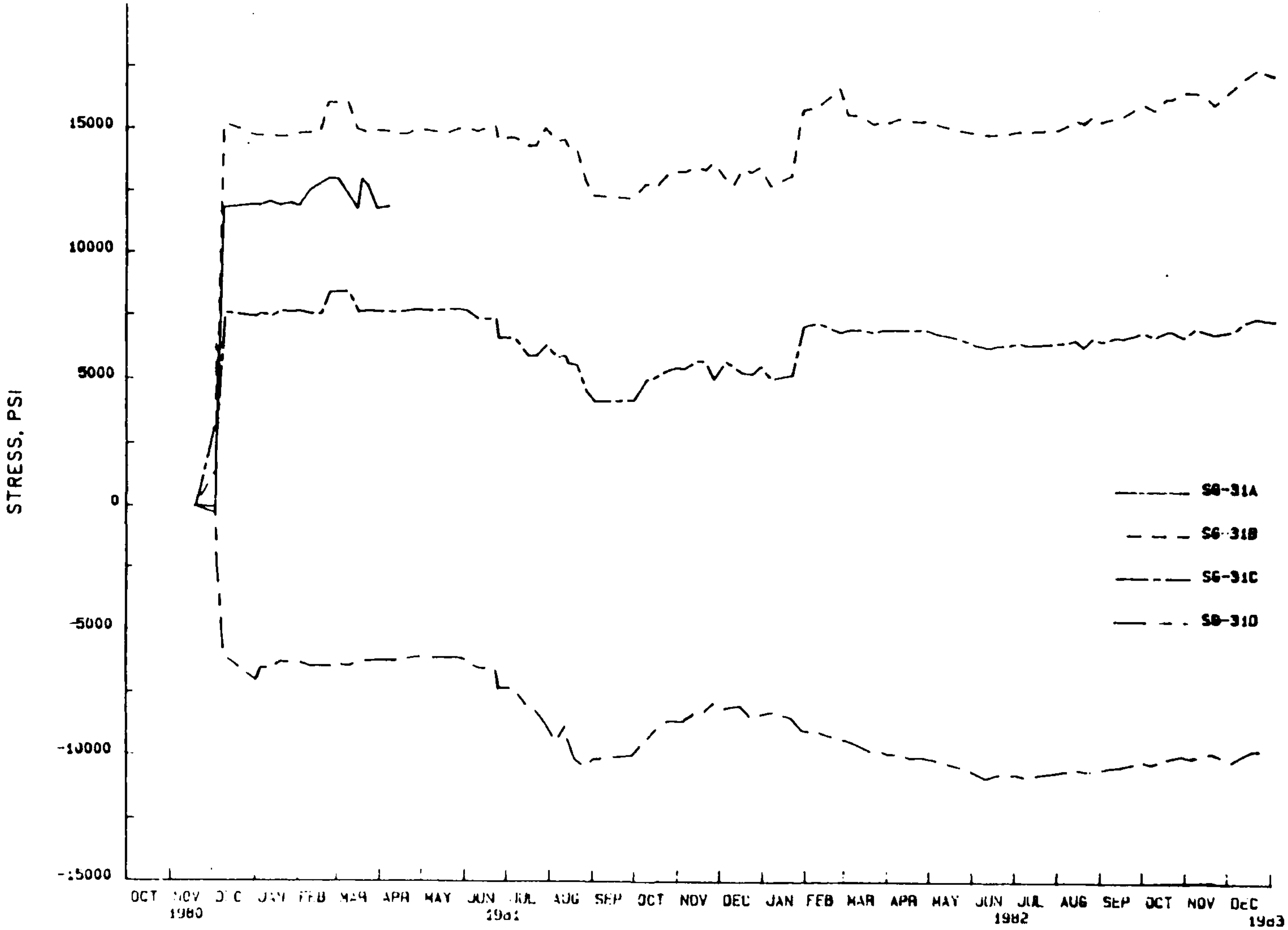


Figure A-14. (B-196) Time history plots of sheet pile strain gauges in dam (SG-31) 1981-1982.

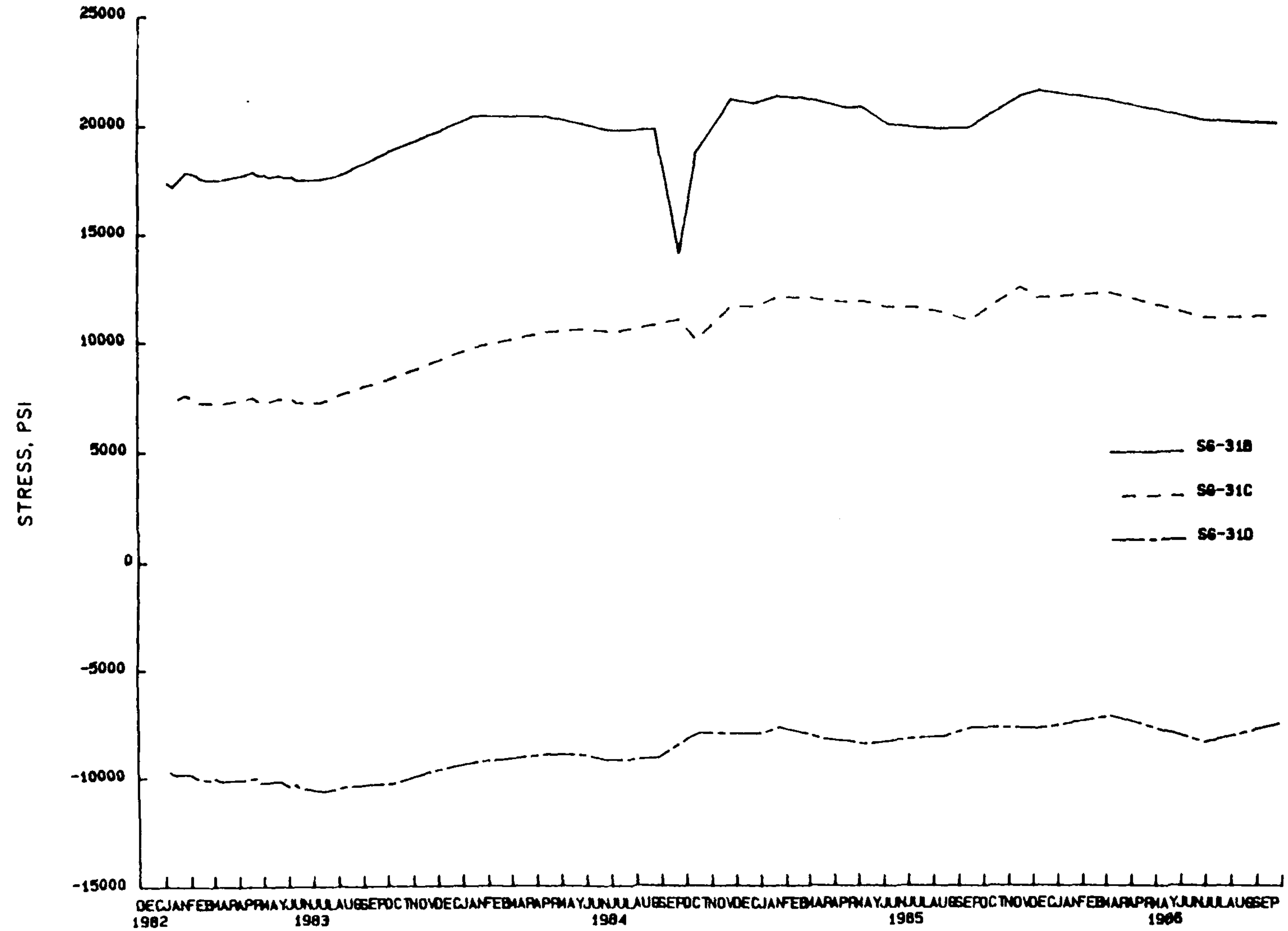


Figure A-15. (B-197) Time history plots of sheet pile strain gauges in dam (SG-31) 1983-1986.

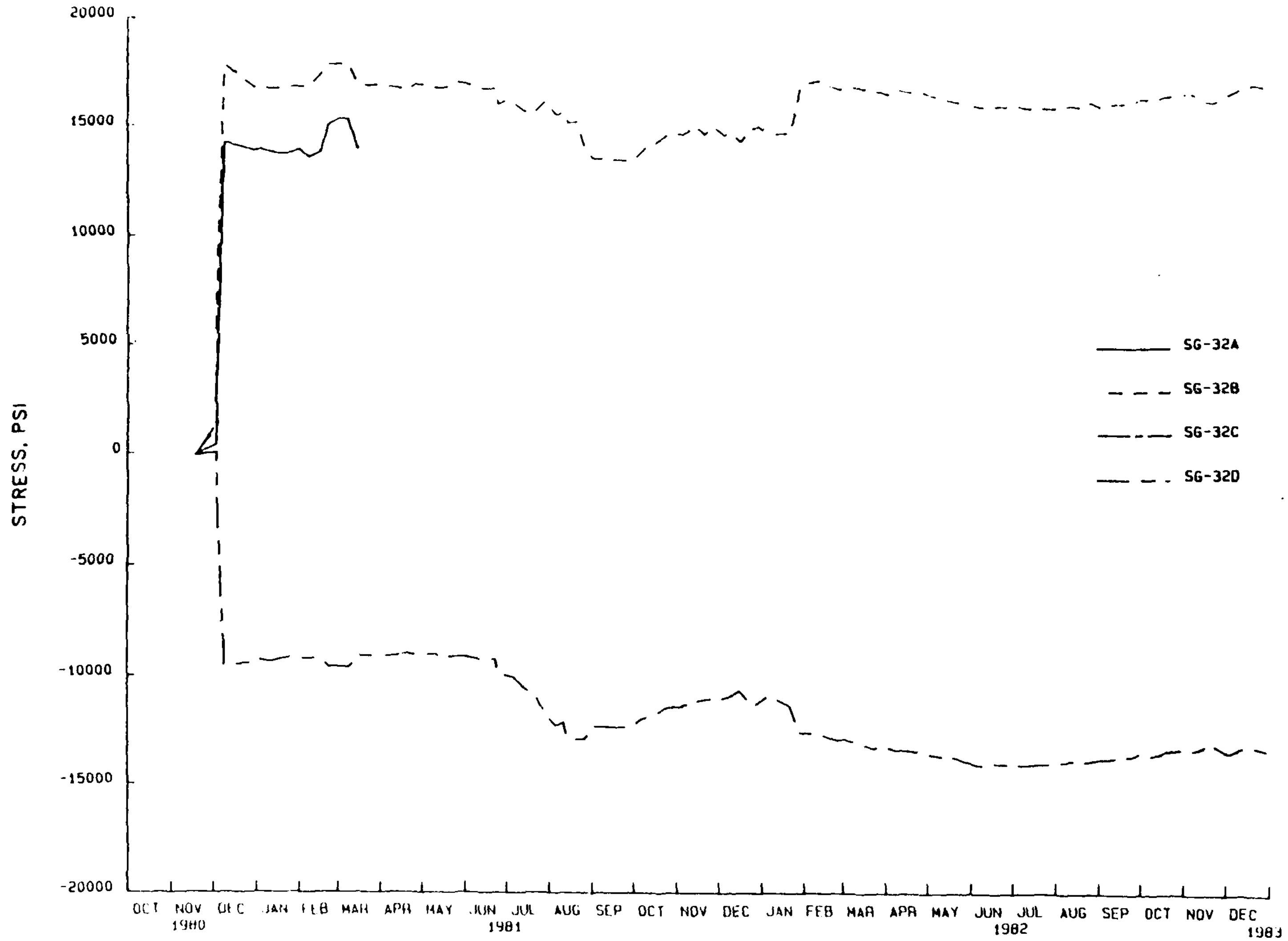


Figure A-16. (B-198) Time history plots of sheet pile strain gauges in dam (SG-32) 1981-1982.

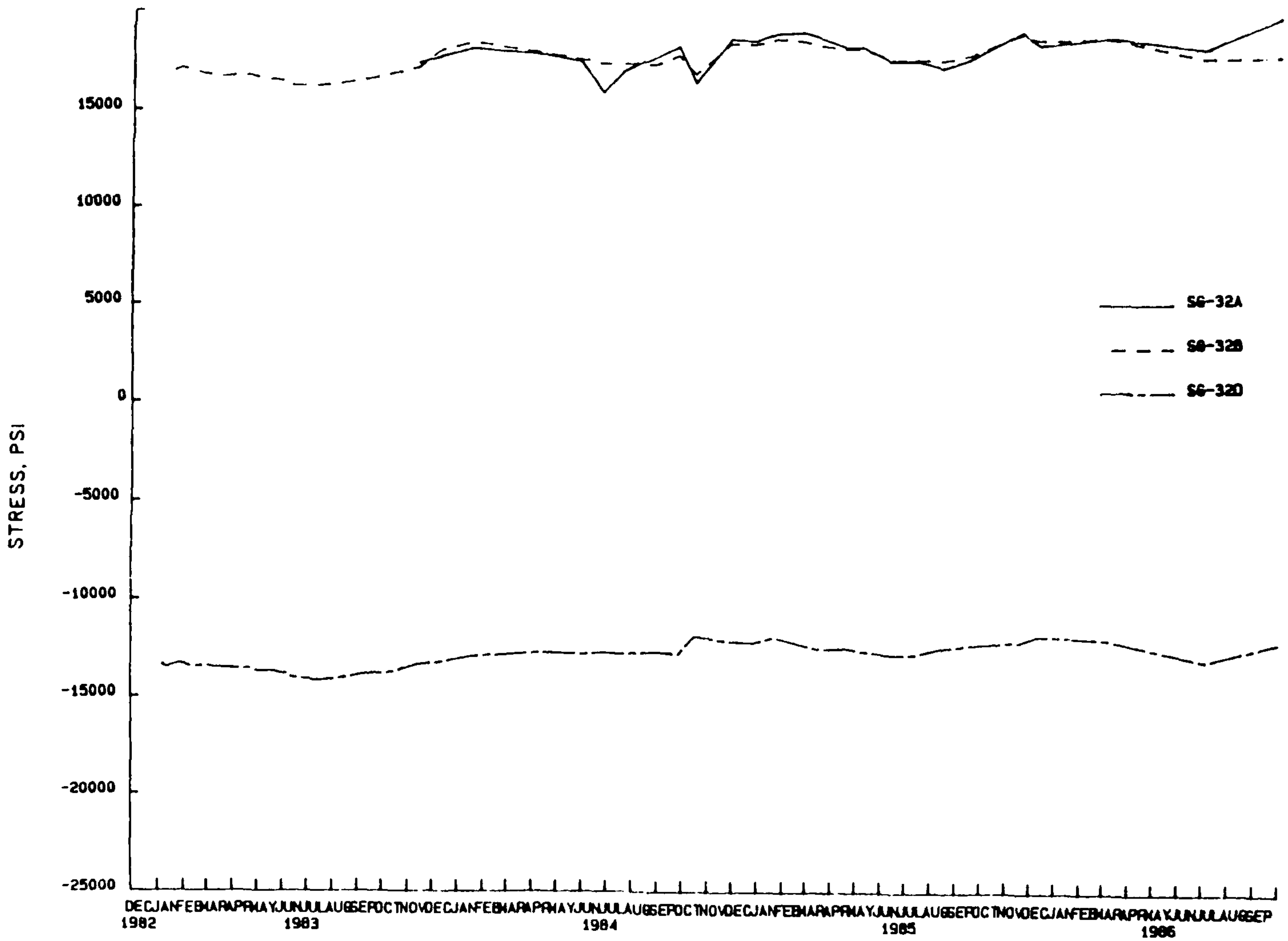


Figure A-17. (B-199) Time history plots of sheet pile strain gauges in dam (SG-32) 1 33-1986.

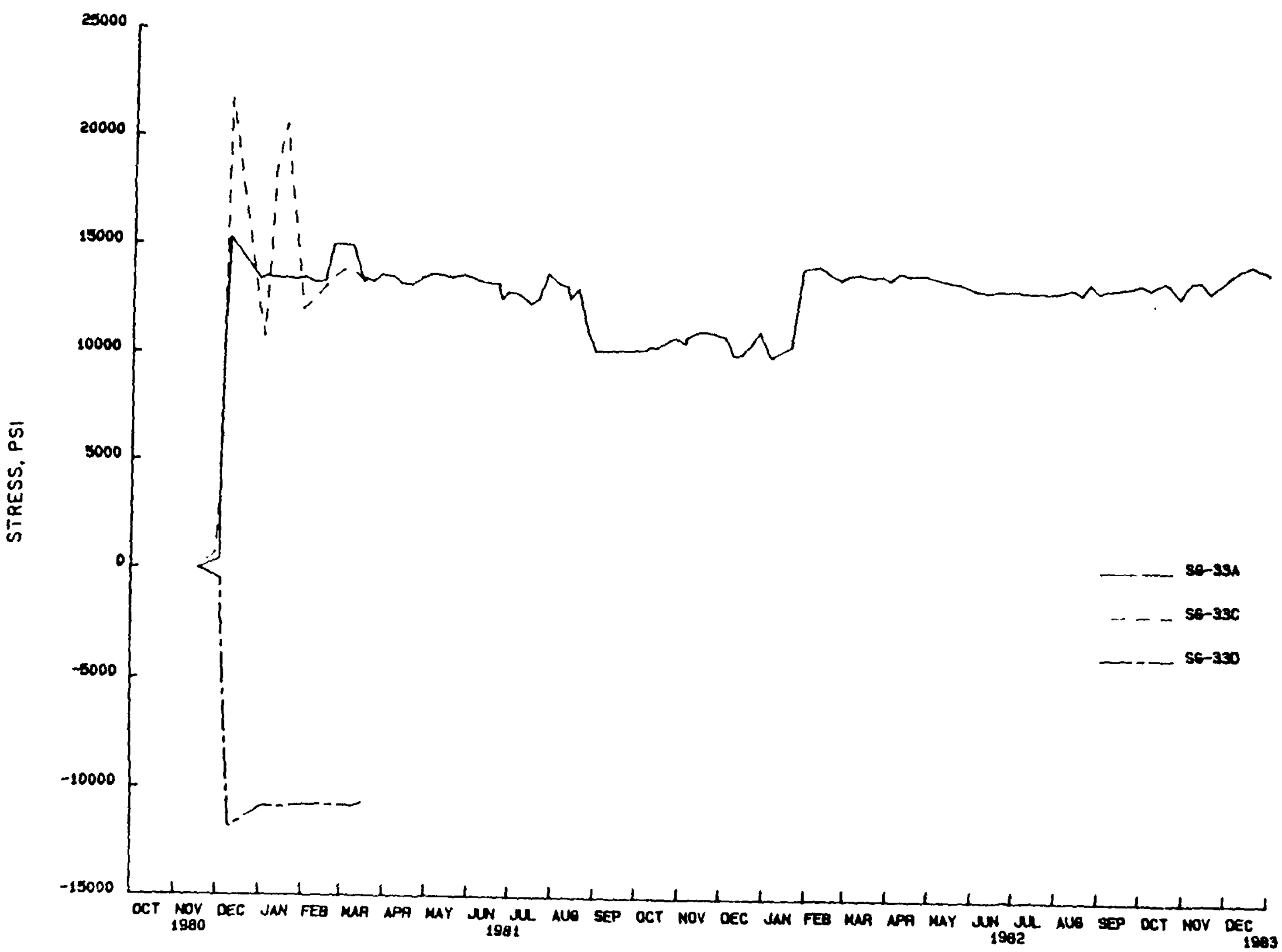


Figure A-18. (B-200) Time history plots of sheet pile strain gauges in dam (SG-33) 1981-1982.

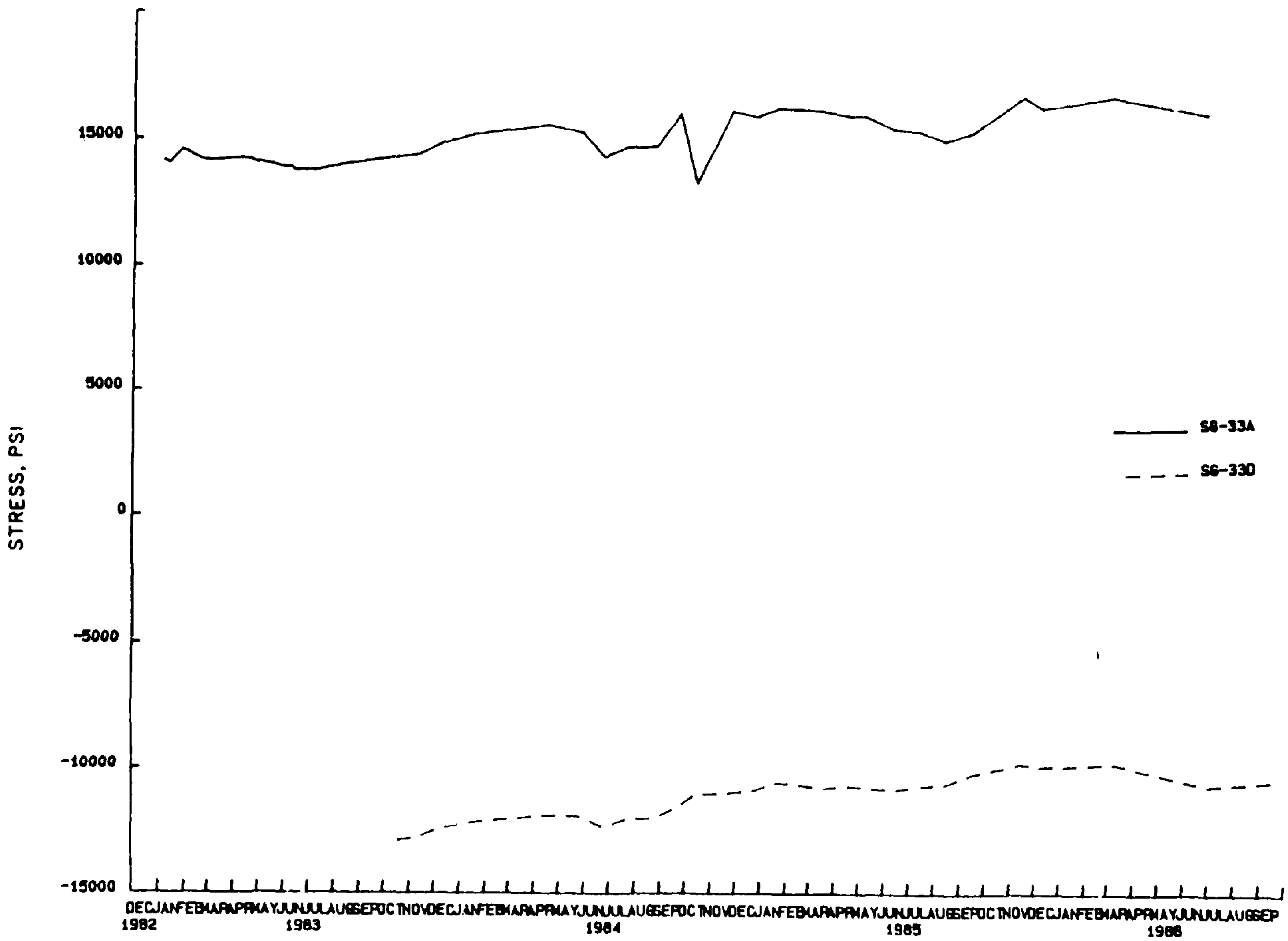


Figure A-19. (B-201) Time history plots of sheet pile strain gauges in dam (SG-33) 1983-1986.

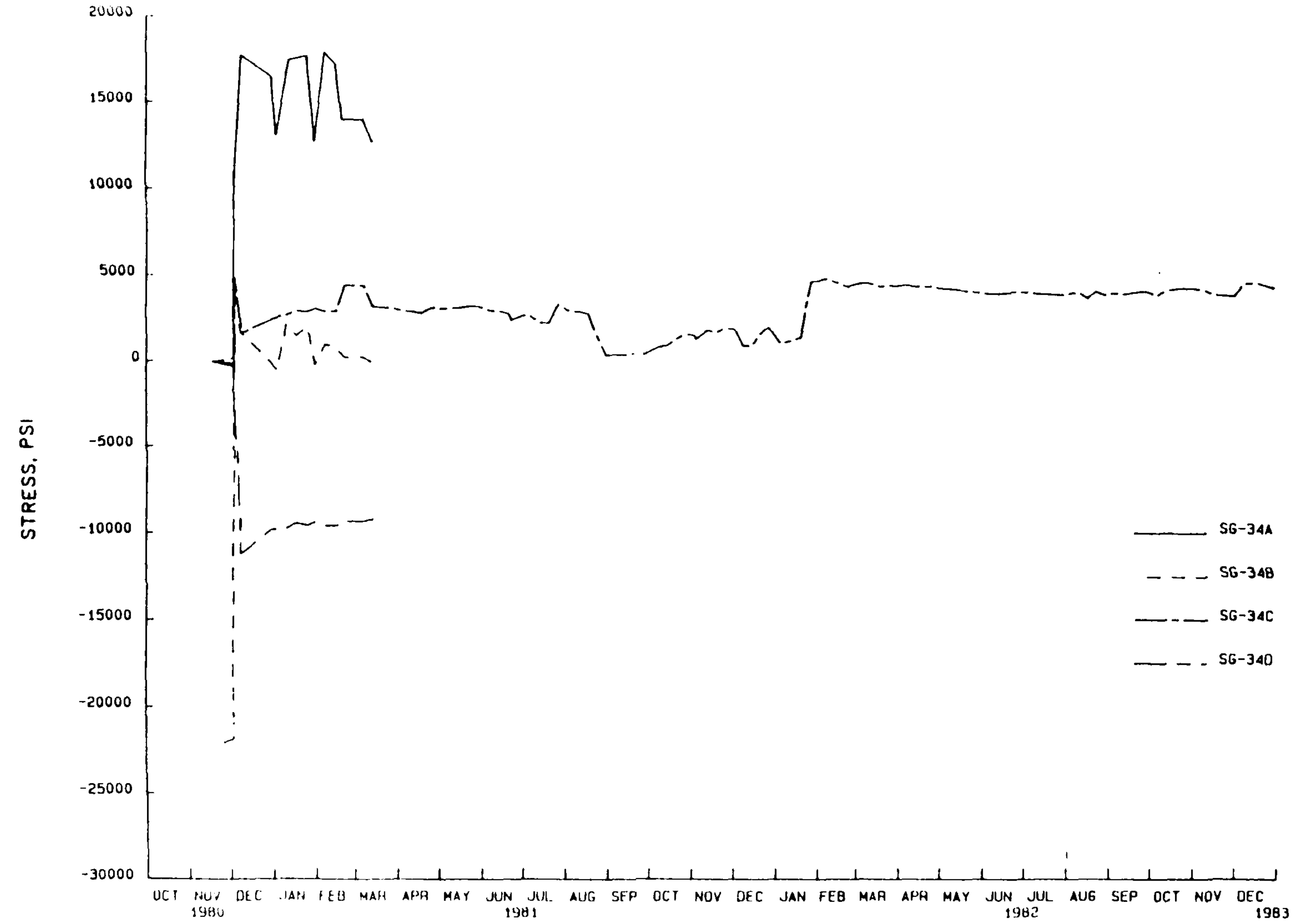


Figure A-20. (B-202) Time history plots of sheet pile strain gauges in dam (SG-34) 1981-1982.

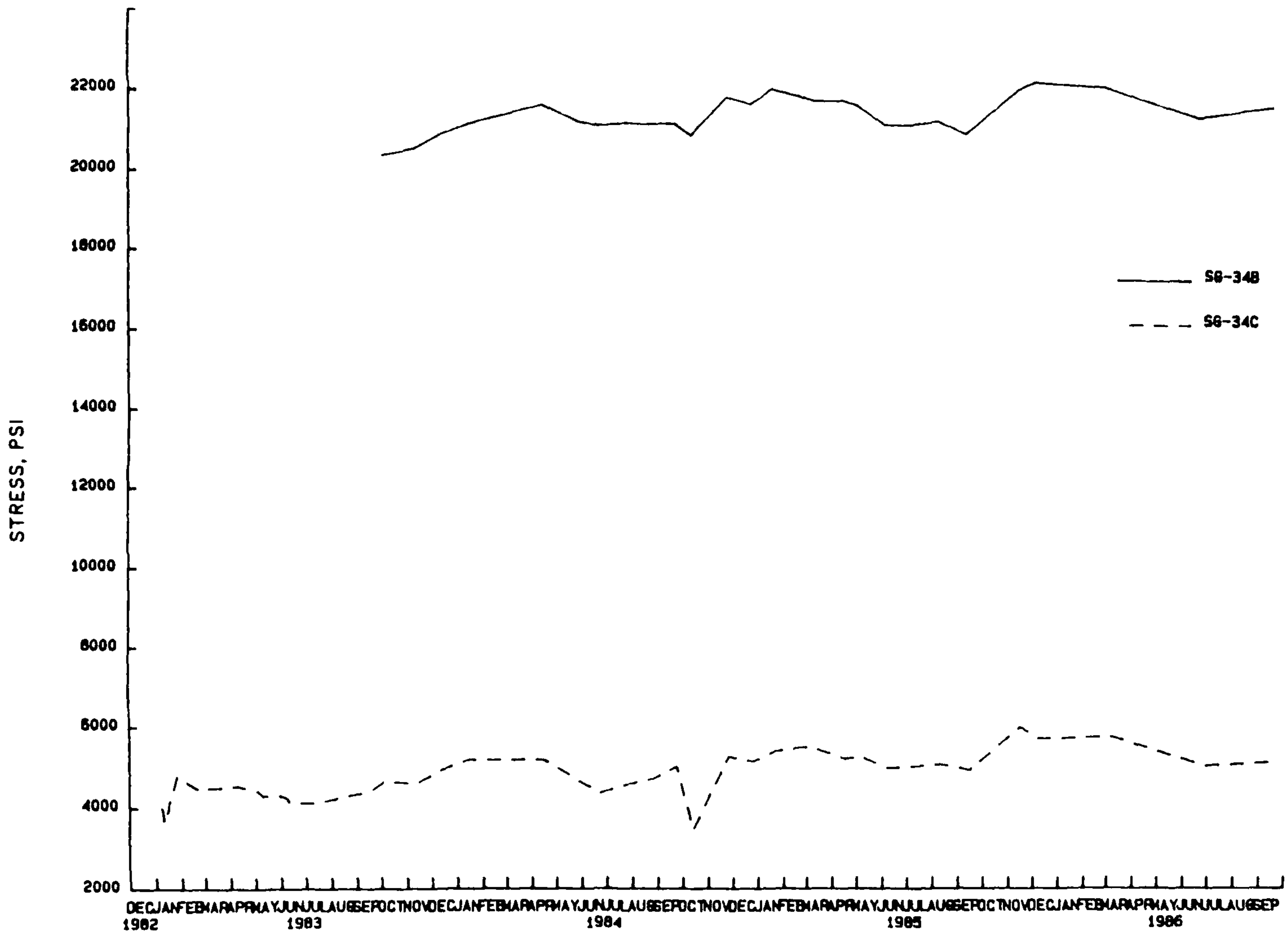


Figure A-21. (B-203) Time history plots of sheet pile strain gauges in dam (SG-34) 1983-1986.

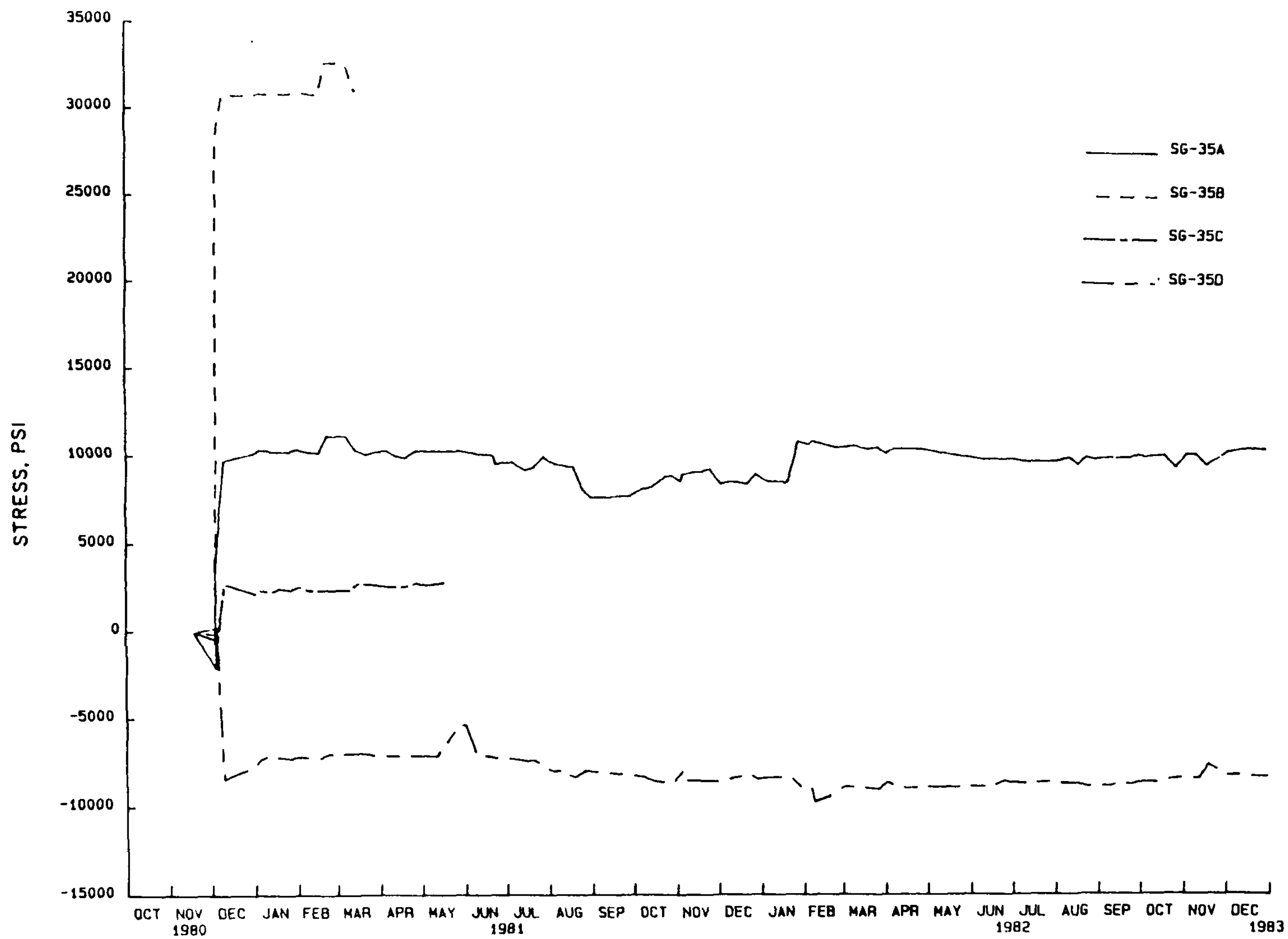


Figure A-22. (B-204) **Time history plots of sheet pile strain gauges in dam (SG-35) 1981-1982.**

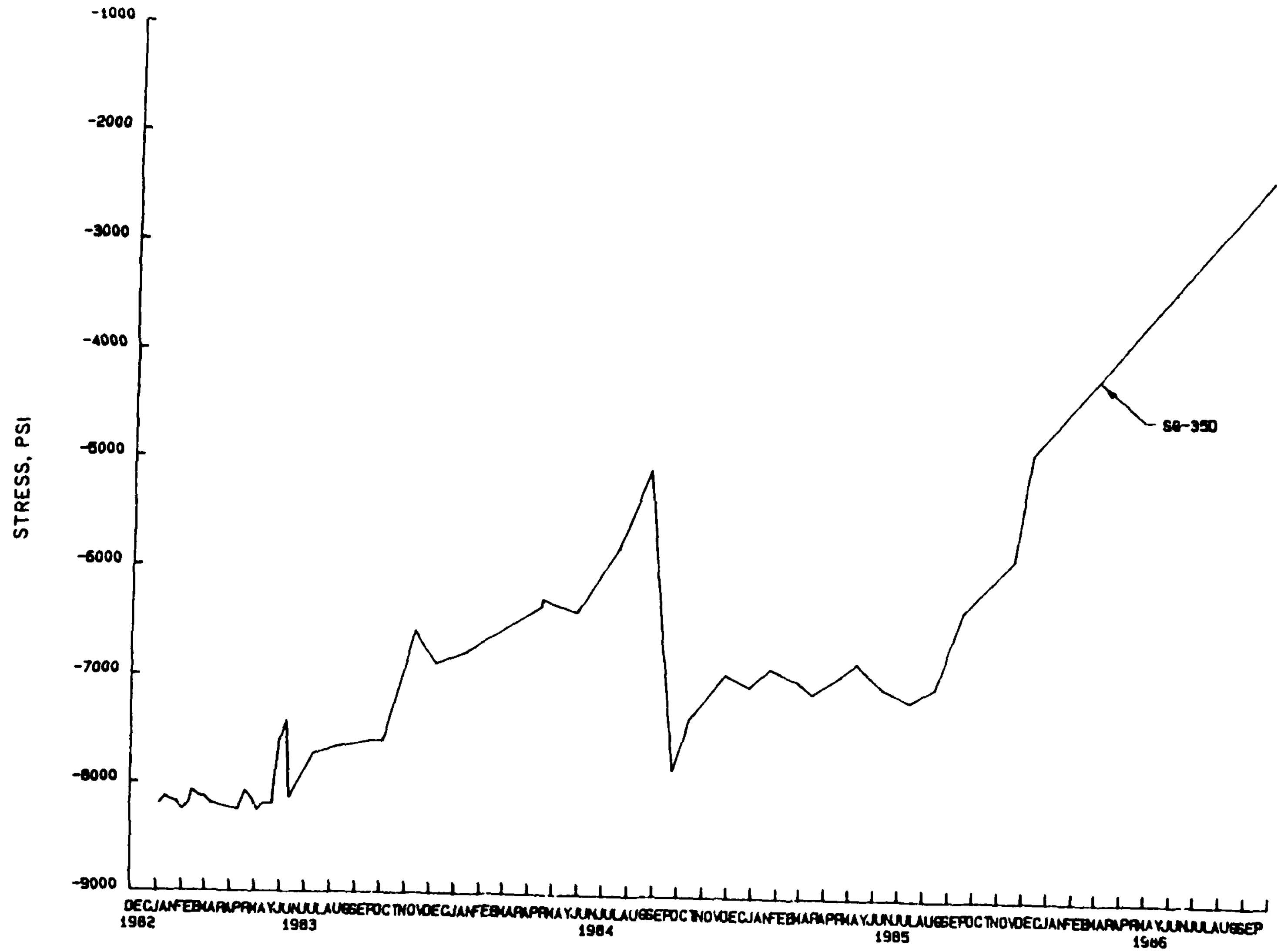


Figure A-23. (B-205) Time history plots of sheet pile strain gauges in dam (SG-35) 1983-1986.

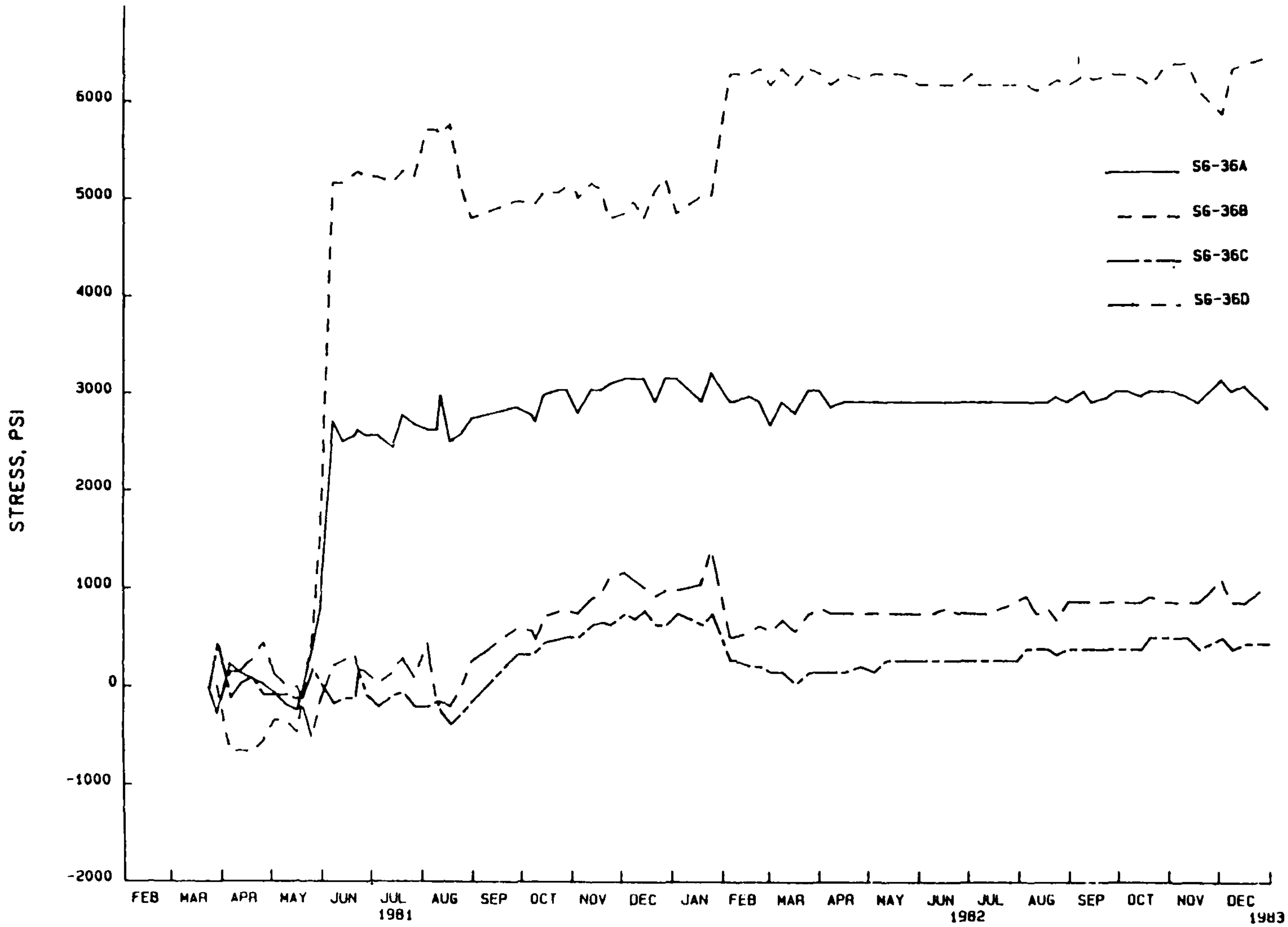


Figure A-24. (B-206) Time history plots of sheet pile strain gauges in dam (SG-36) 1981-1982.

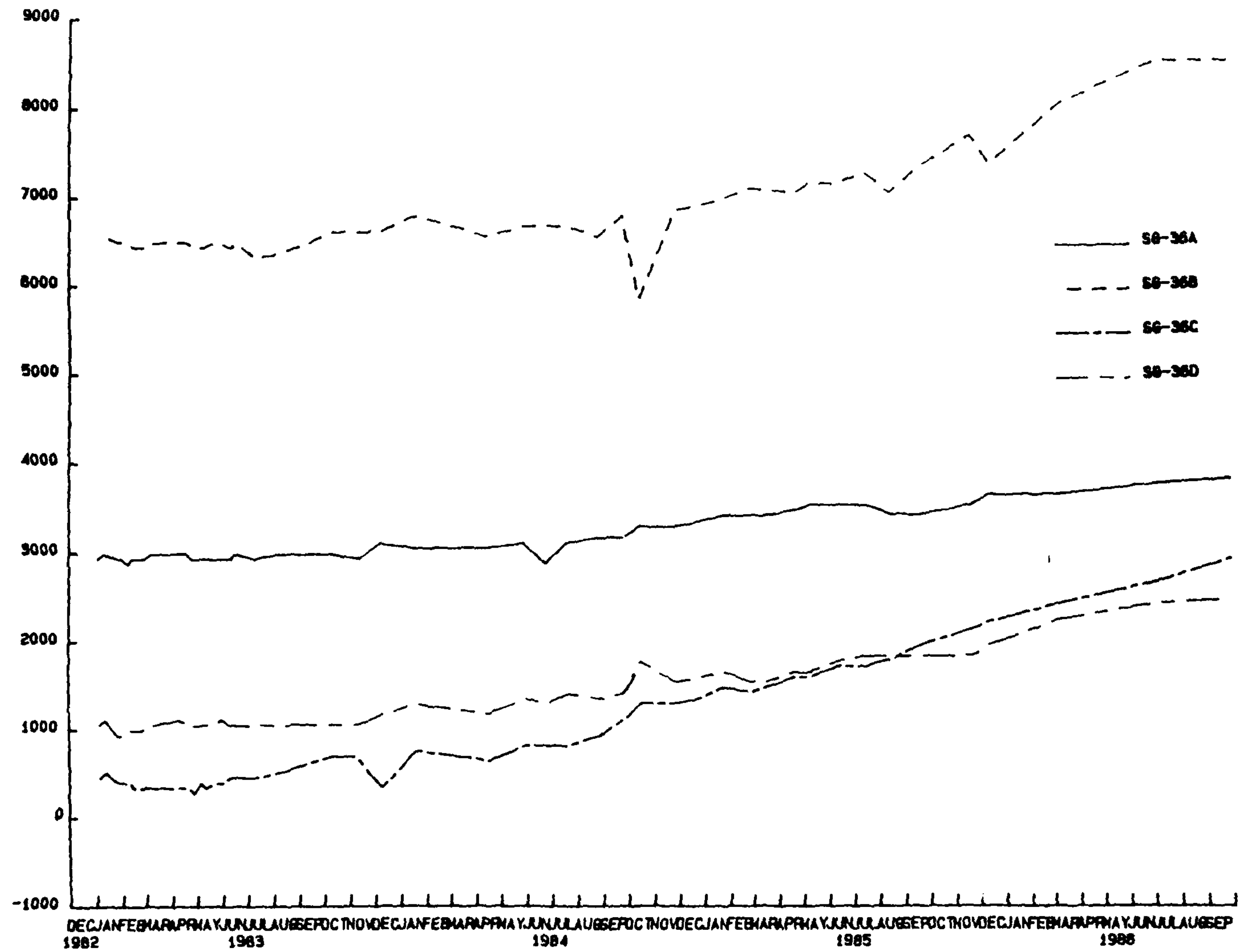


Figure A-25. (B-207) Time history plots of sheet pile strain gauges in dam (SG-36) 1983-1986.

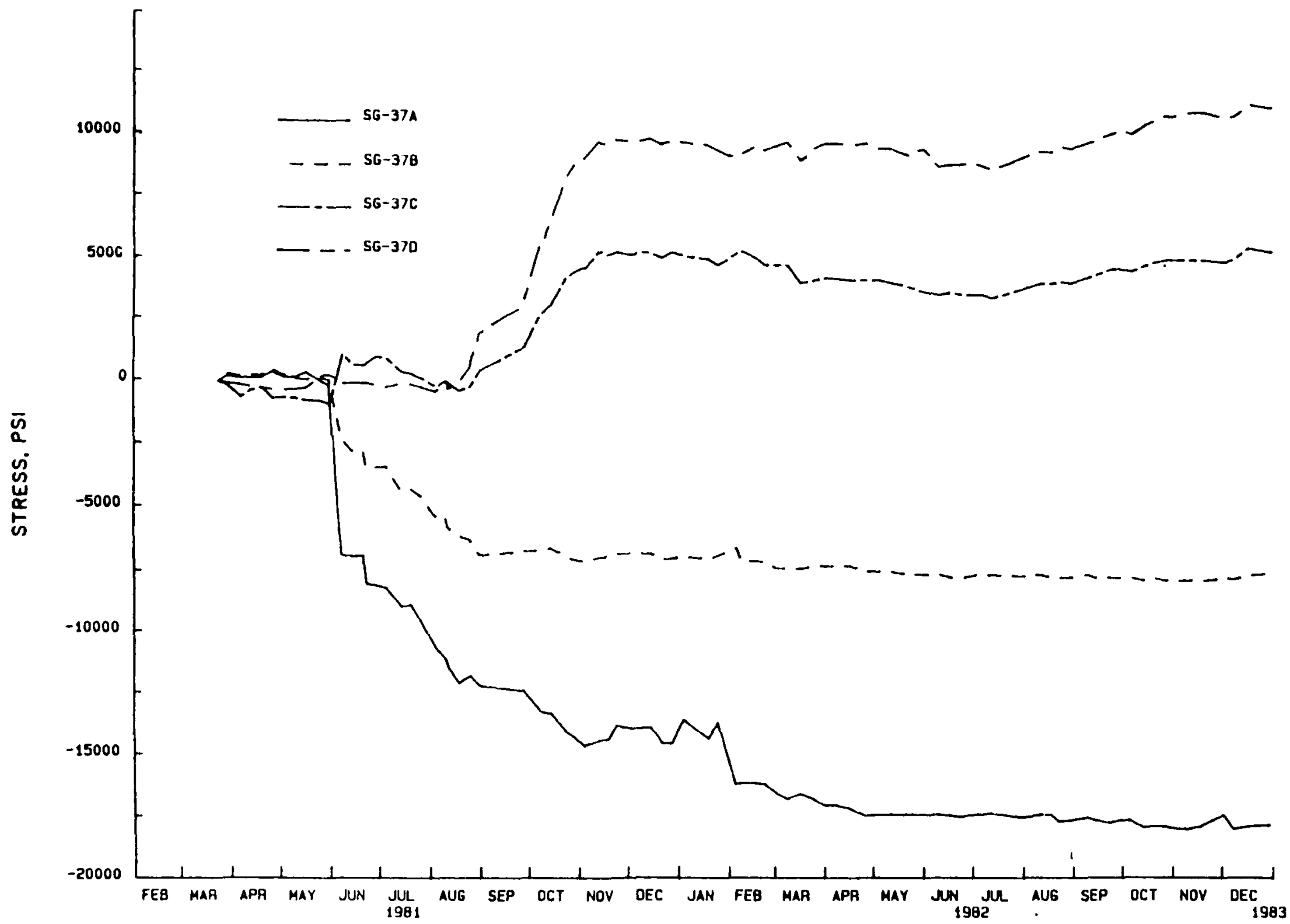


Figure A-26. (B-208) Time history plots of sheet pile strain gauges in dam (SG-37) 1981-1982.

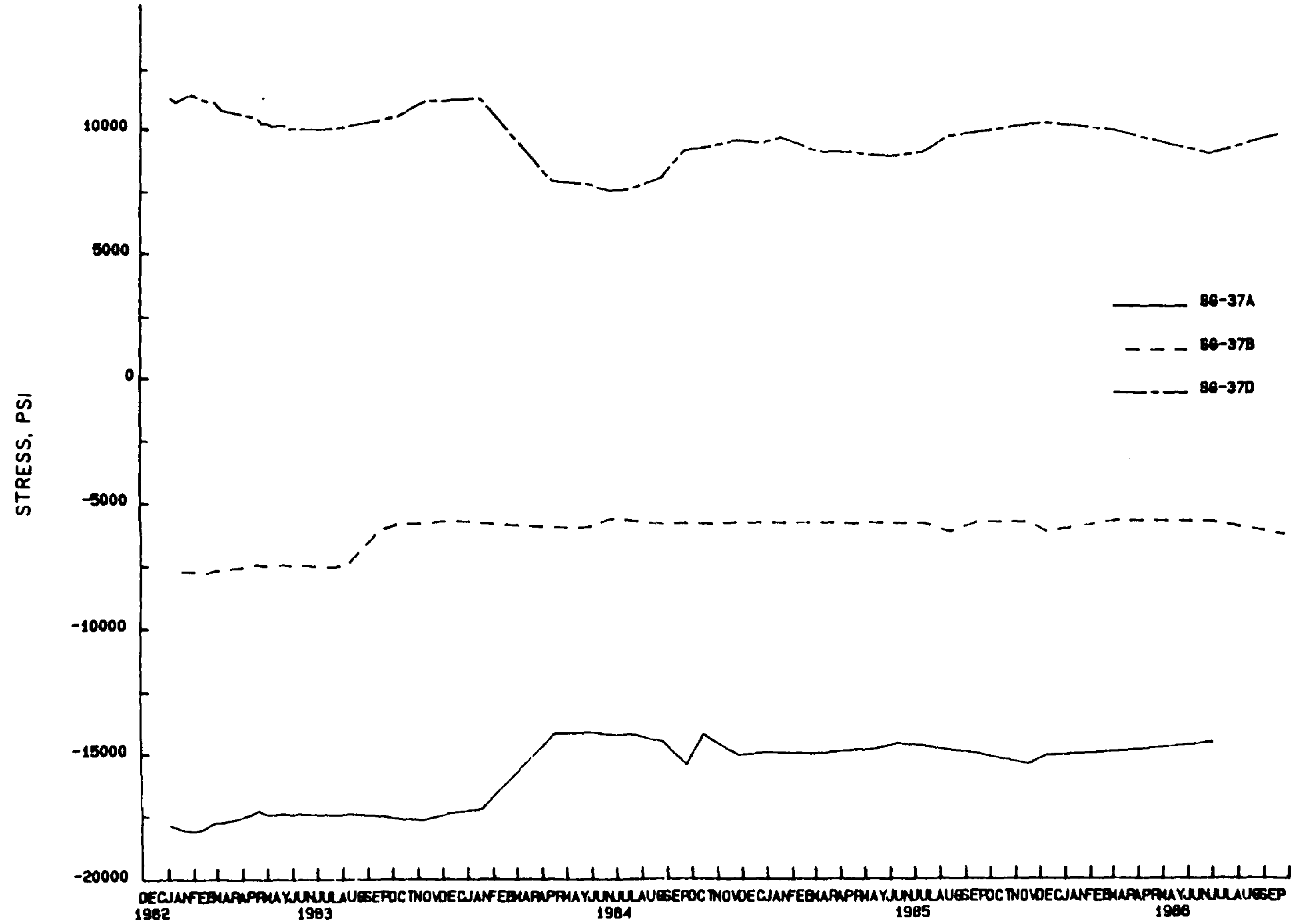


Figure A-27. (B-209) Time history plots of sheet pile strain gauges in dam (SG-37) 1983-1986.

Figure A-28. (B-210) Time history plots of sheet pile strain gauges in dam (SG-38) 1981-1982.

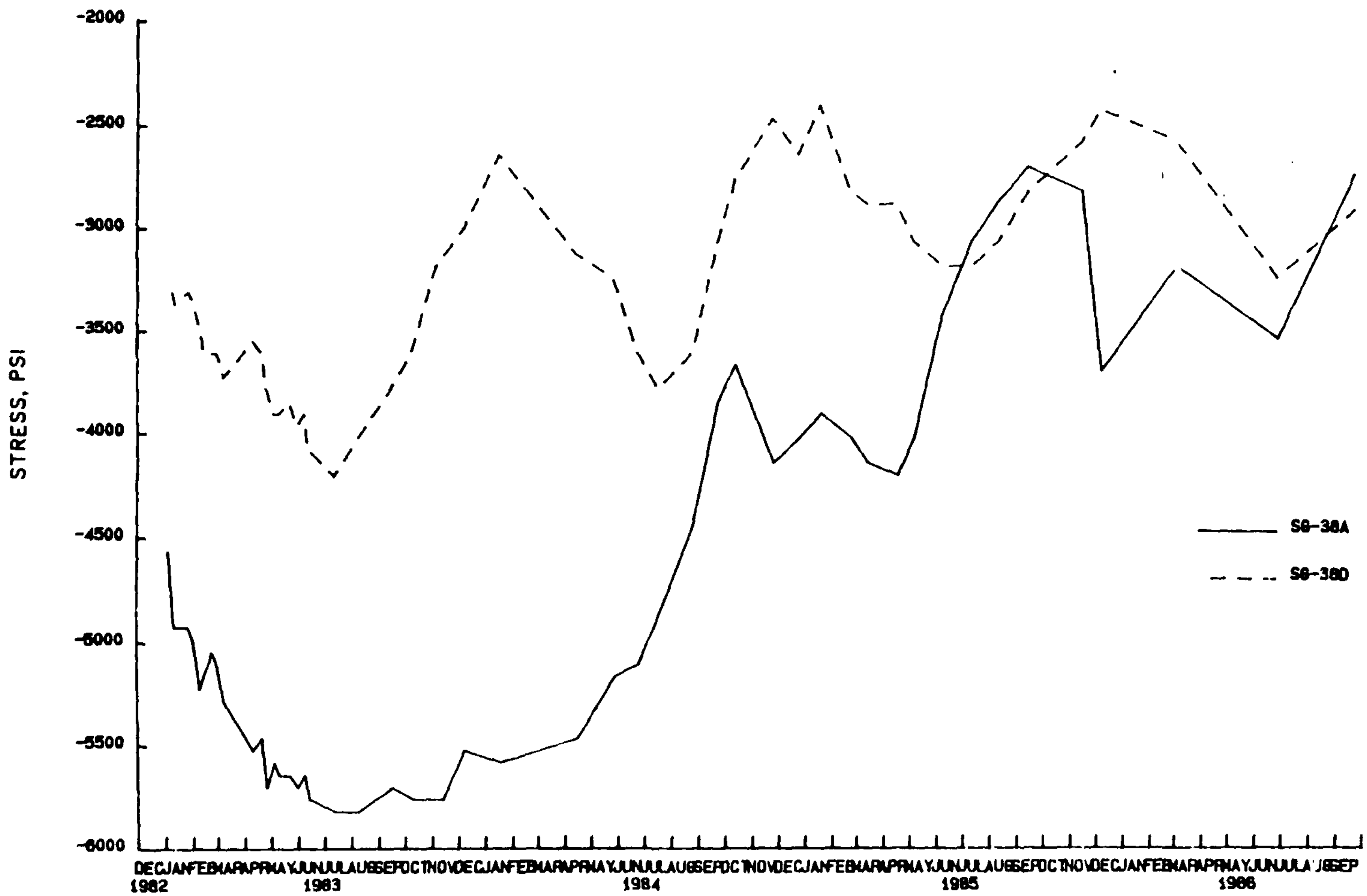


Figure A-29. (B-211) Time history plots of sheet pile strain gauges in dam (SG-38) 1983-1986.

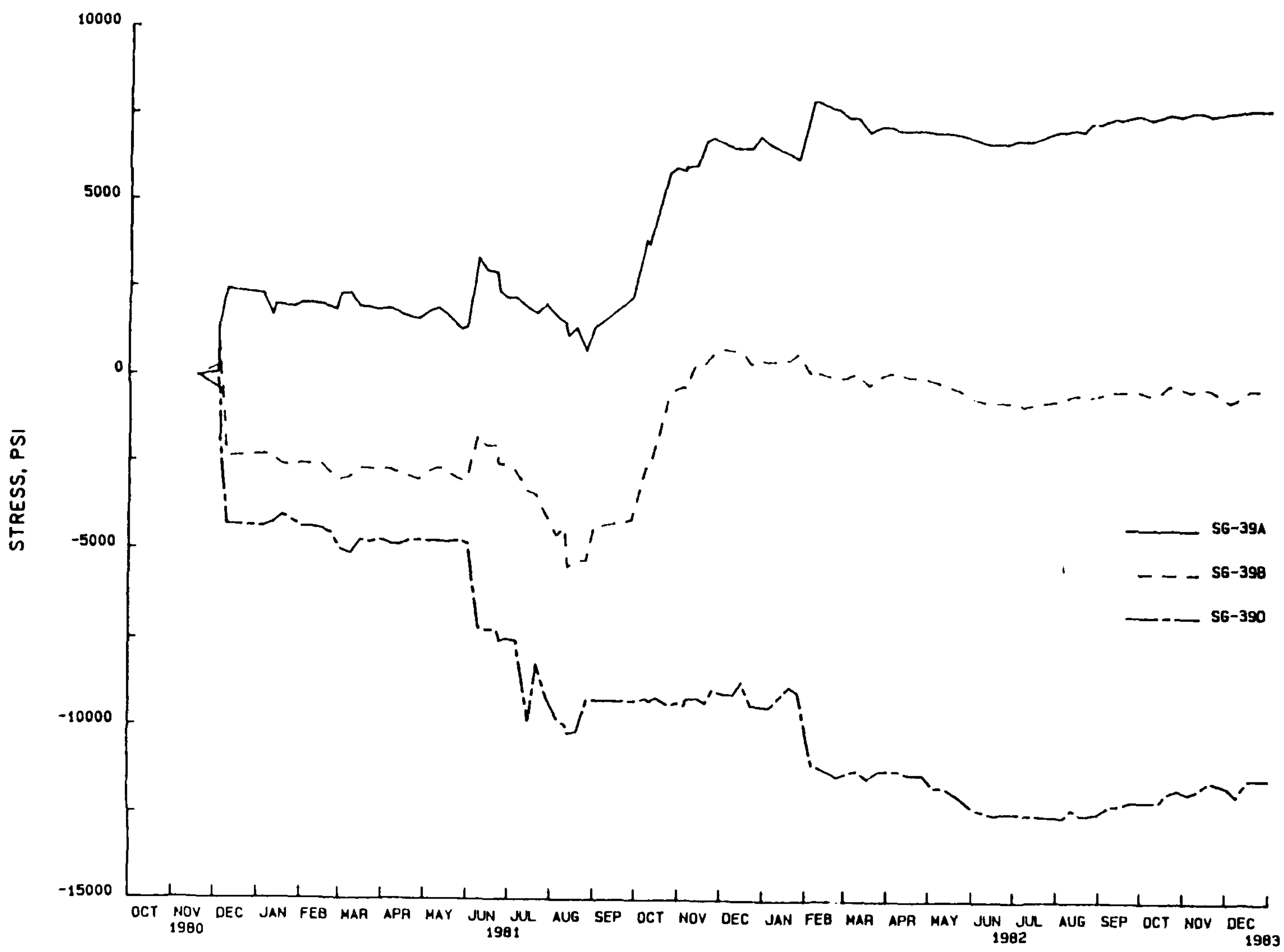


Figure A-30. (B-212) Time history plots of sheet pile strain gauges in dam (SG-39) 1981-1982.

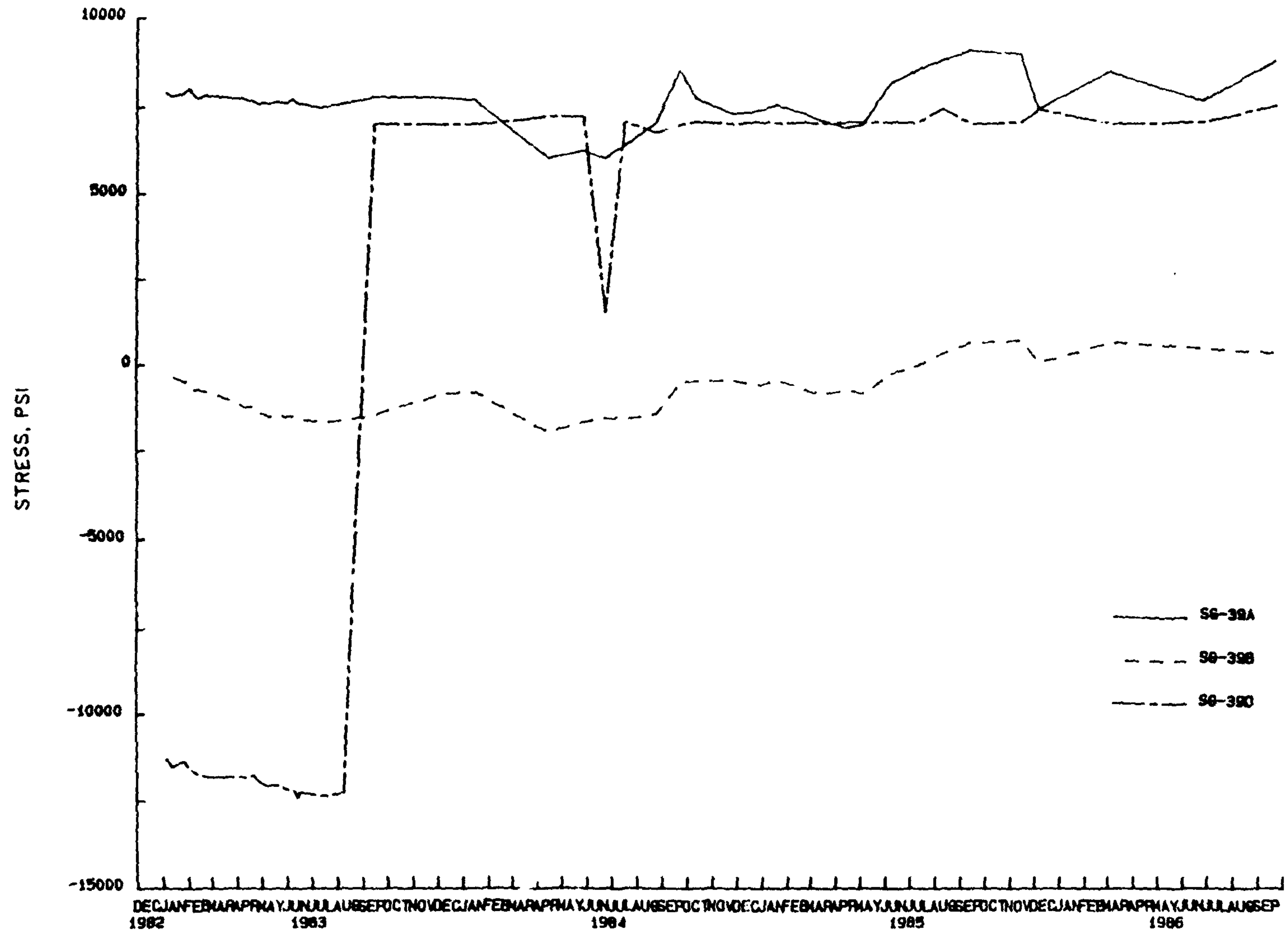


Figure A-31. (B-213) Time history plots of sheet pile strain gauges in dam (SG-39) 1983-1986.

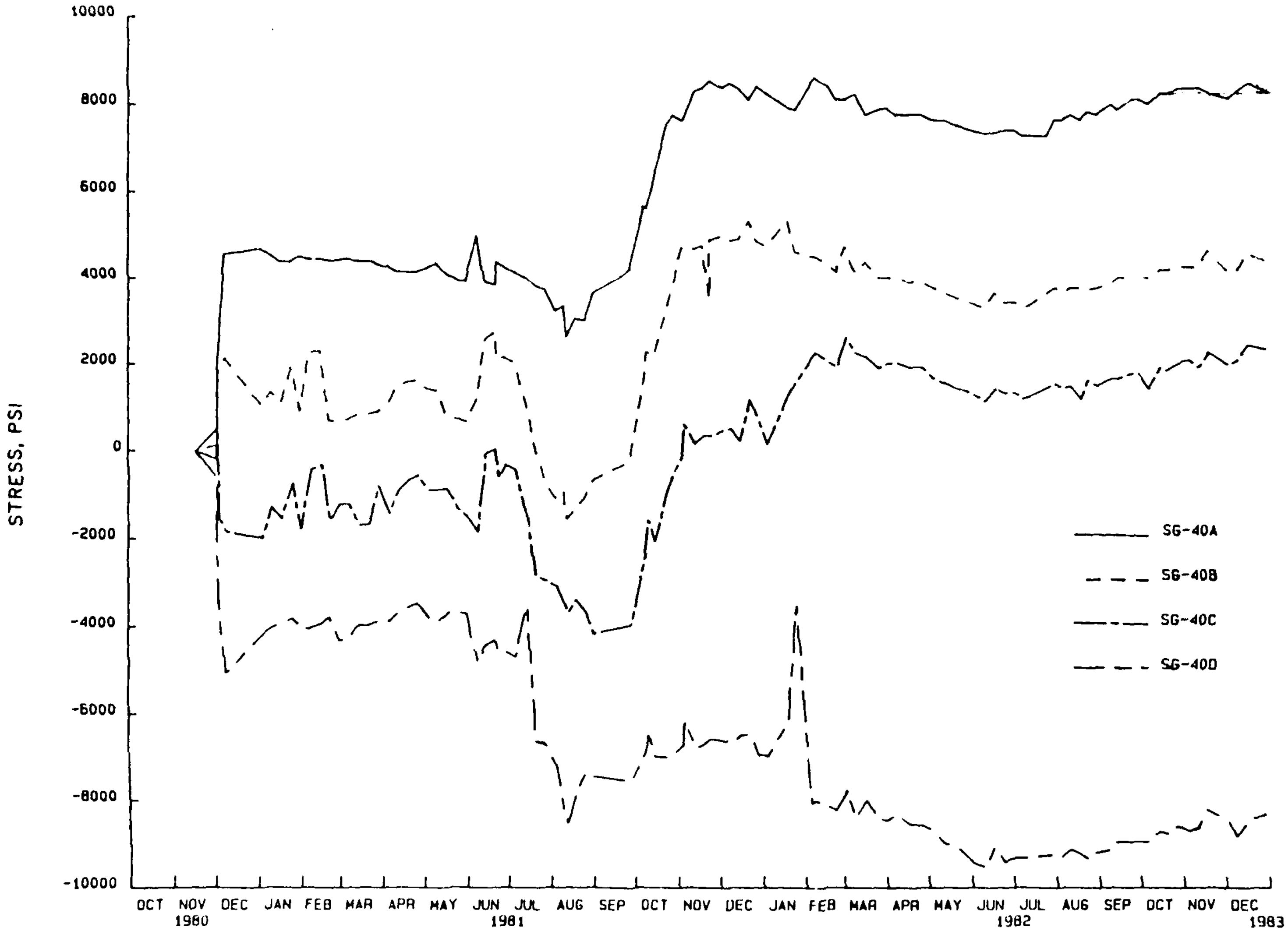


Figure A-32. (B-214) Time history plots of sheet pile strain gauges in dam (SG-40) 1981-1982.

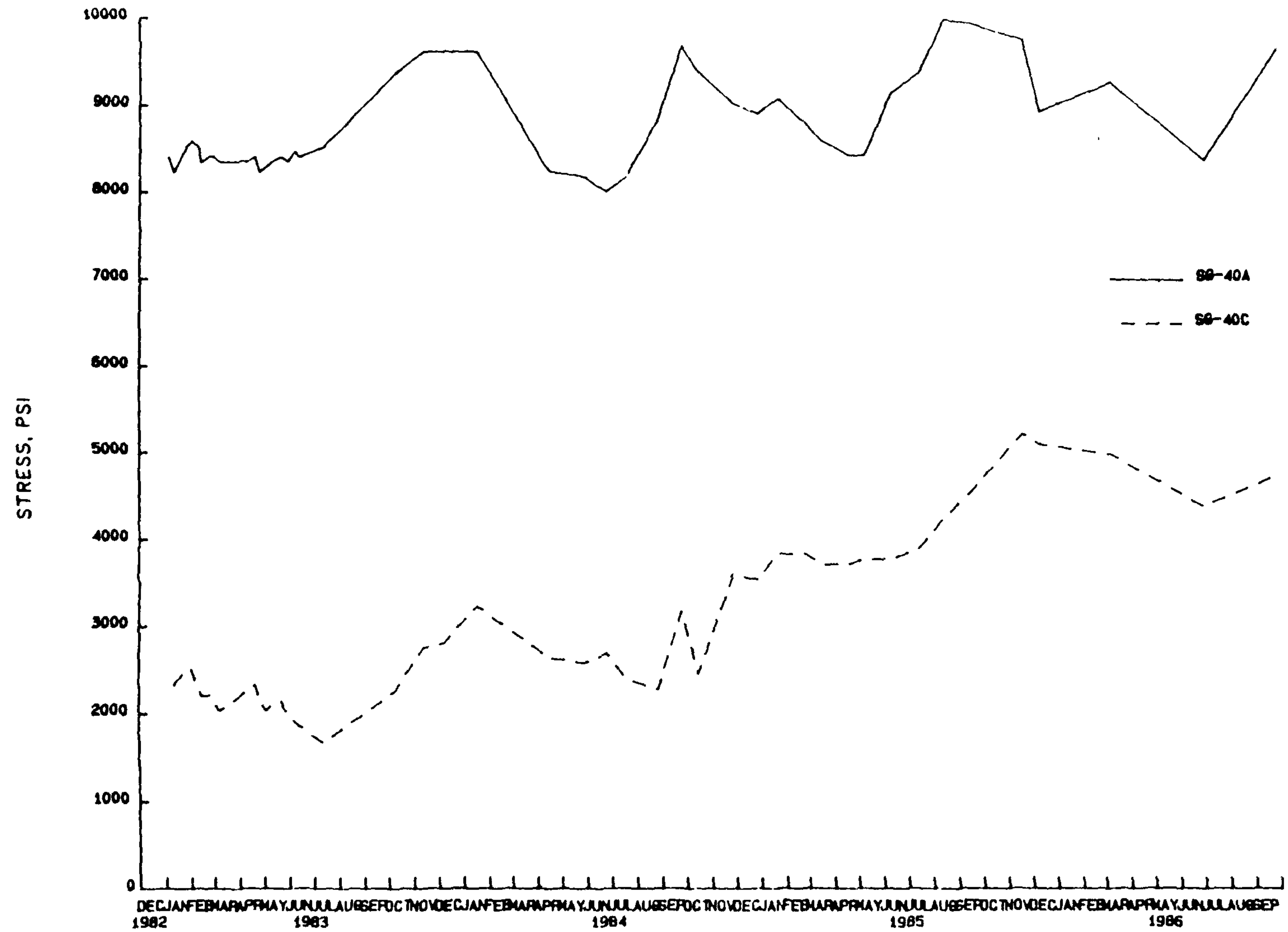


Figure A-33. (B-215) Time history plots of sheet pile strain gauges in dam (*SG-40*) 1983-1986.

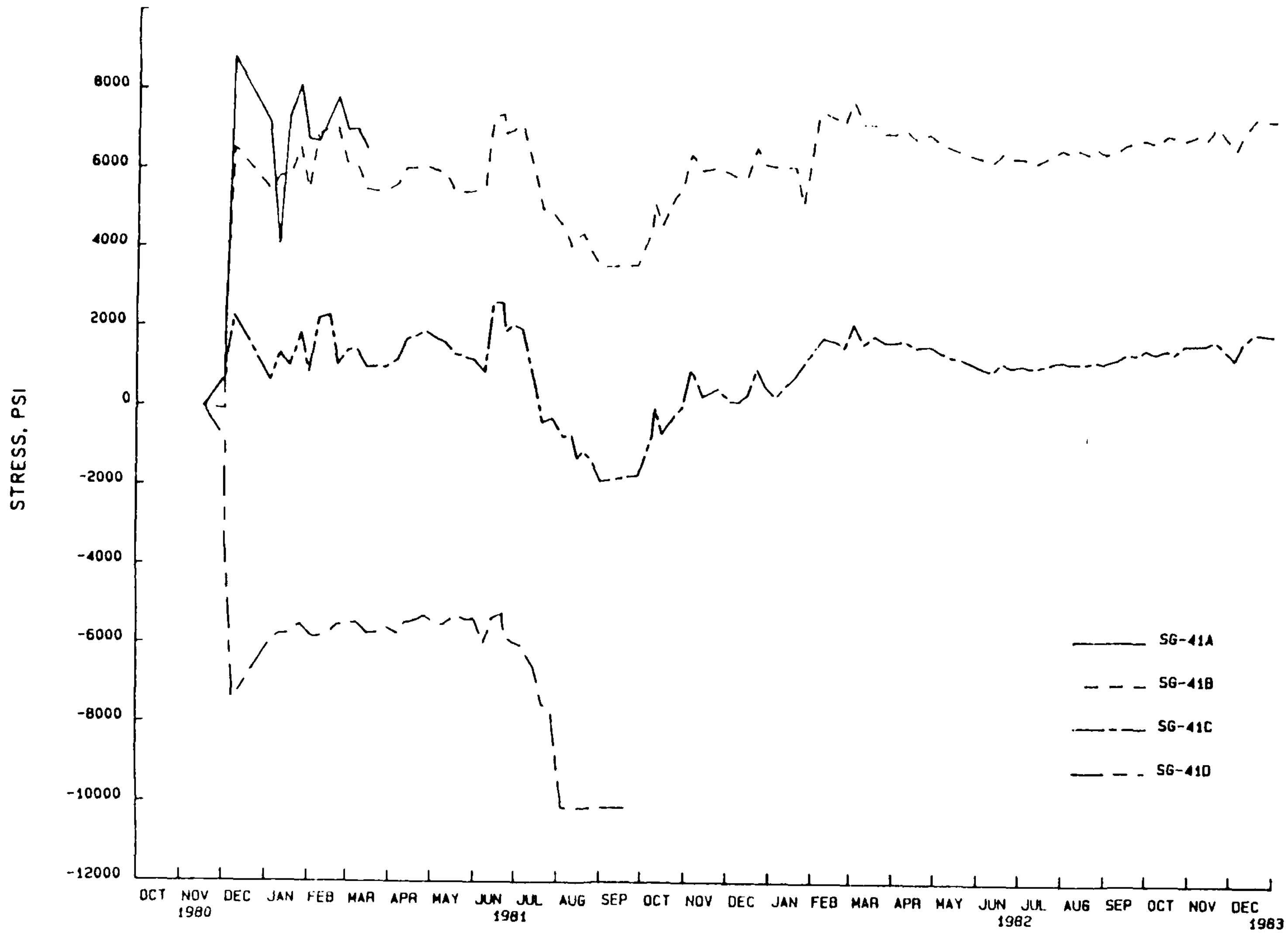


Figure A-34. (B-216) Time history plots of sheet pile strain gaiges in dam (SG-41) 1981-1982.

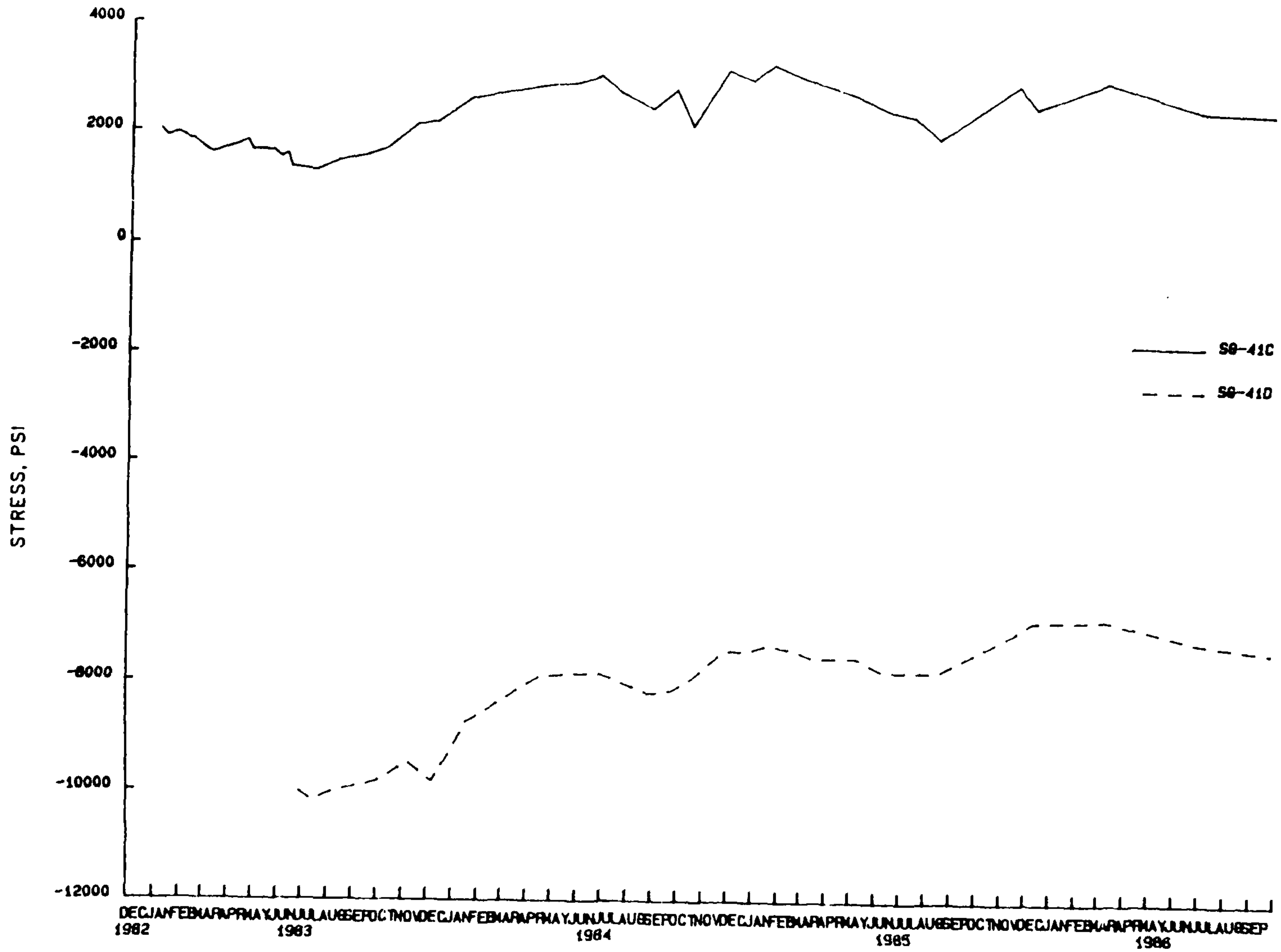


Figure A-35. (B-217) Time history plots of sheet pile strain gauges in dam (SG-41) 1983-1986.

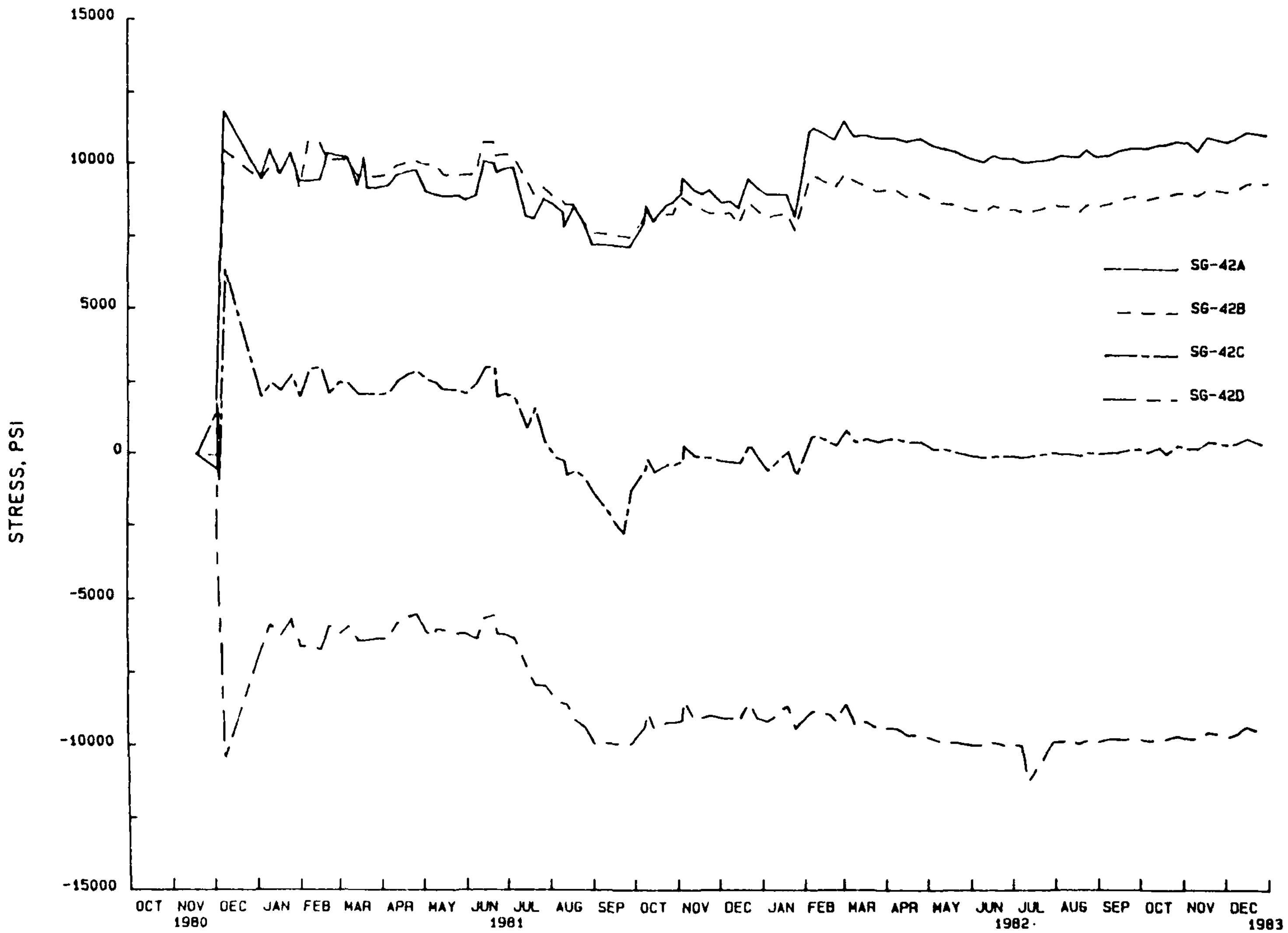


Figure A-36. (B-218) Time history plots of sheet pile strain gauges in dam (SG-42) 1981-1982.

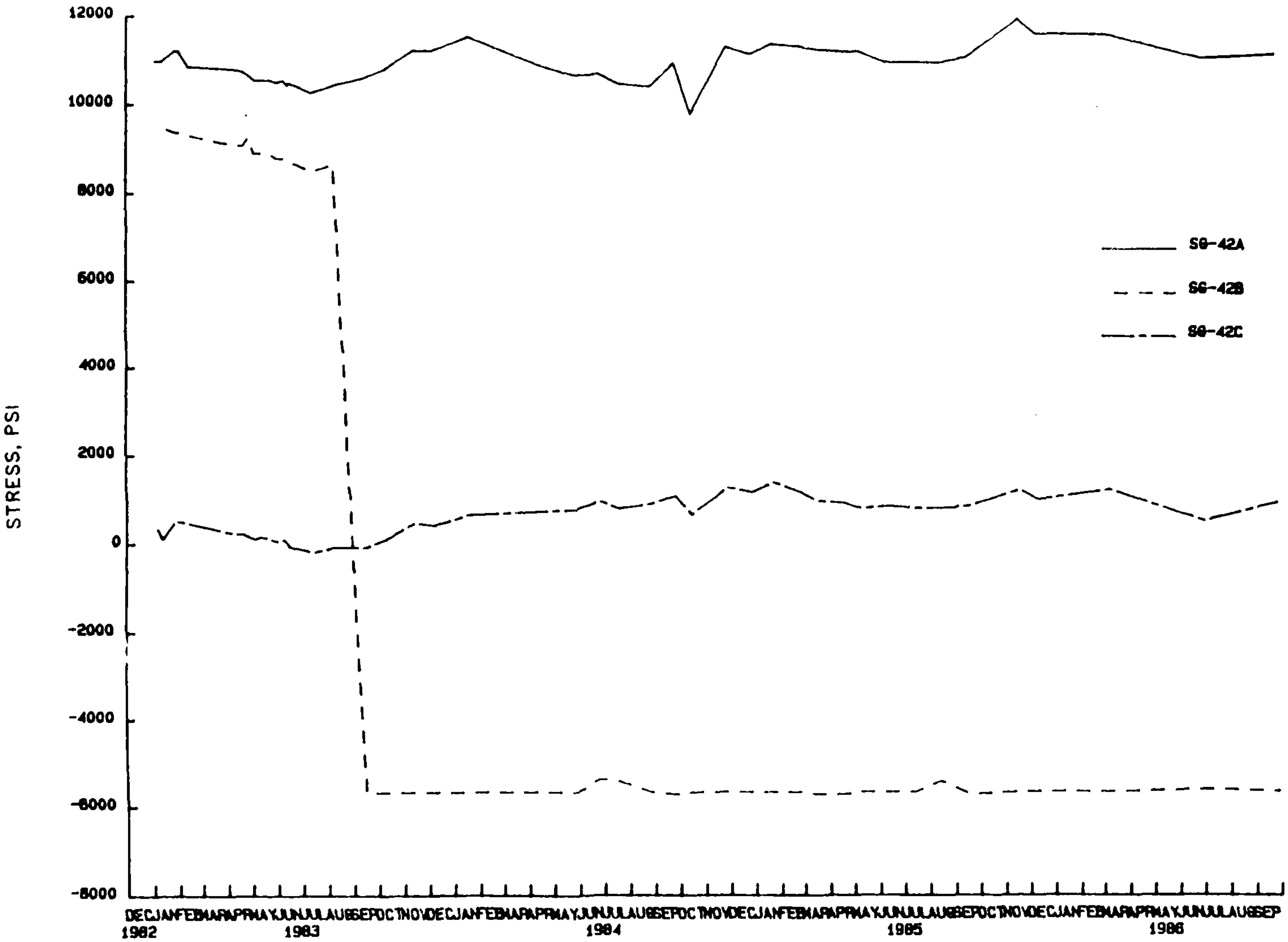


Figure A-37. (B-219) Time history plots of sheet pile strain gauges in dam (SG-42) 1983-1986.

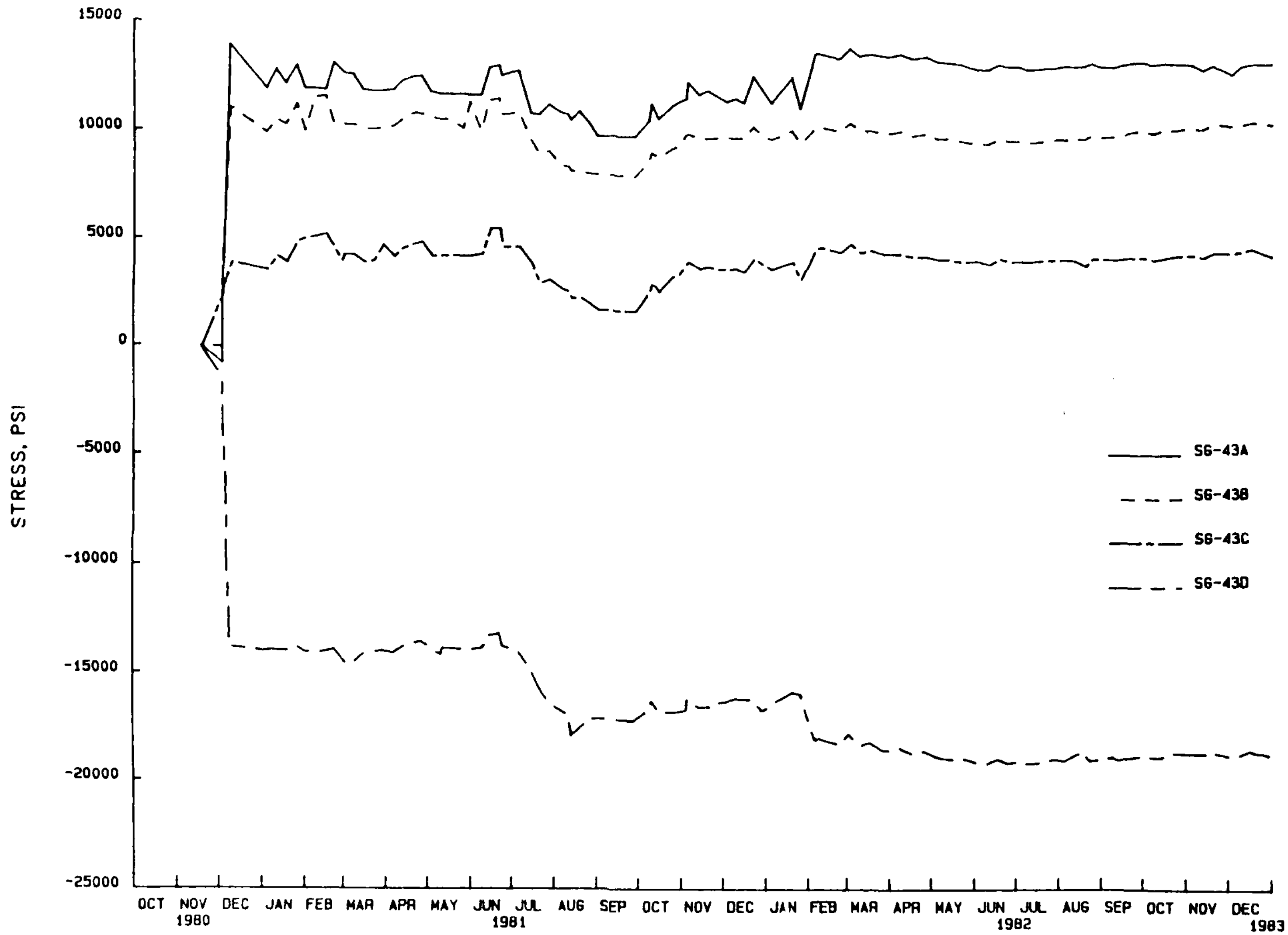


Figure A-38. (B-220) Time history plots of sheet pile strain gauges in dam (SG-43) 1980-1982.

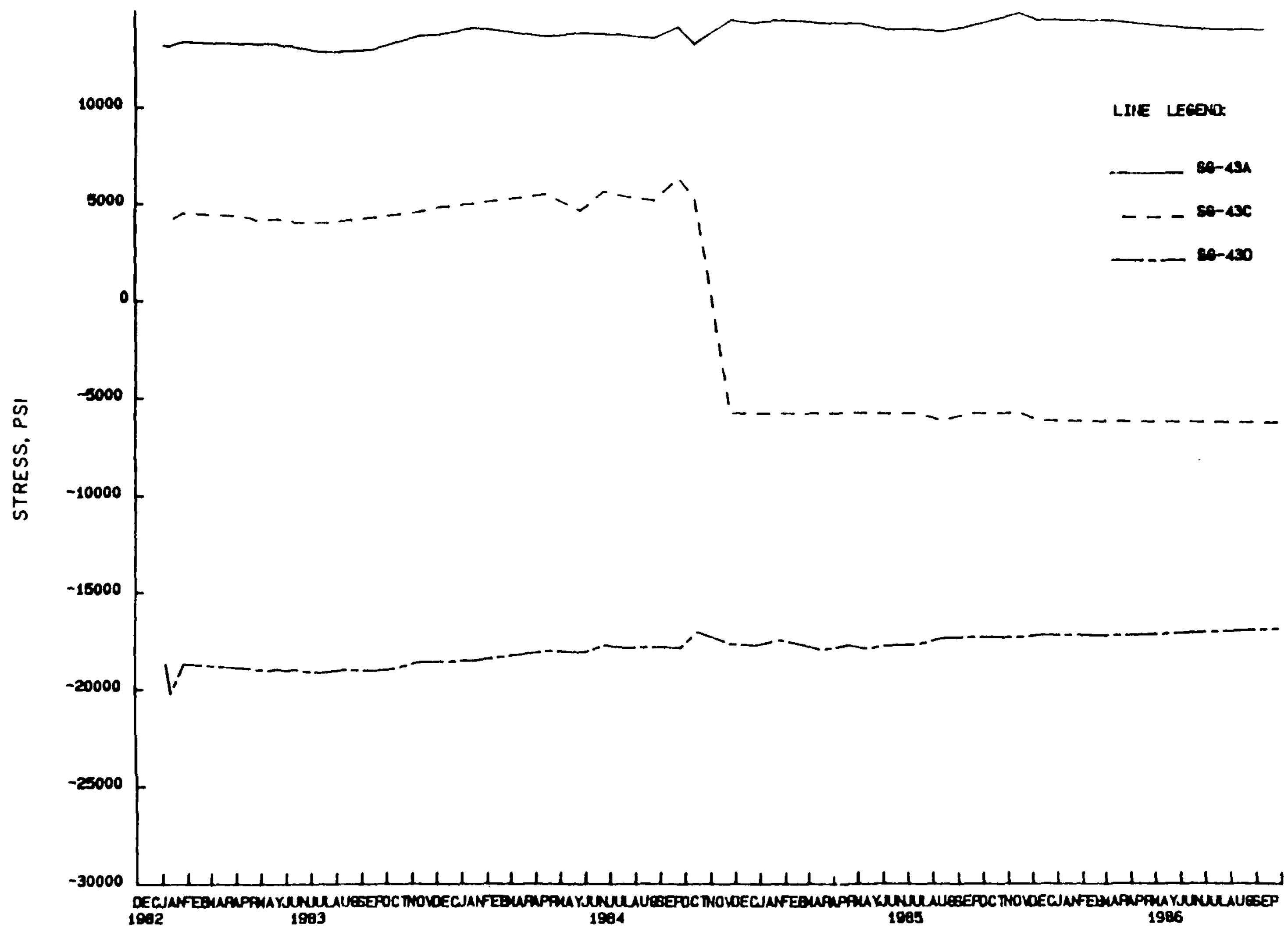


Figure A-39. (B-221) Time history plots of sheet pile strain gauges in dam (SG-43) 1983-1986.

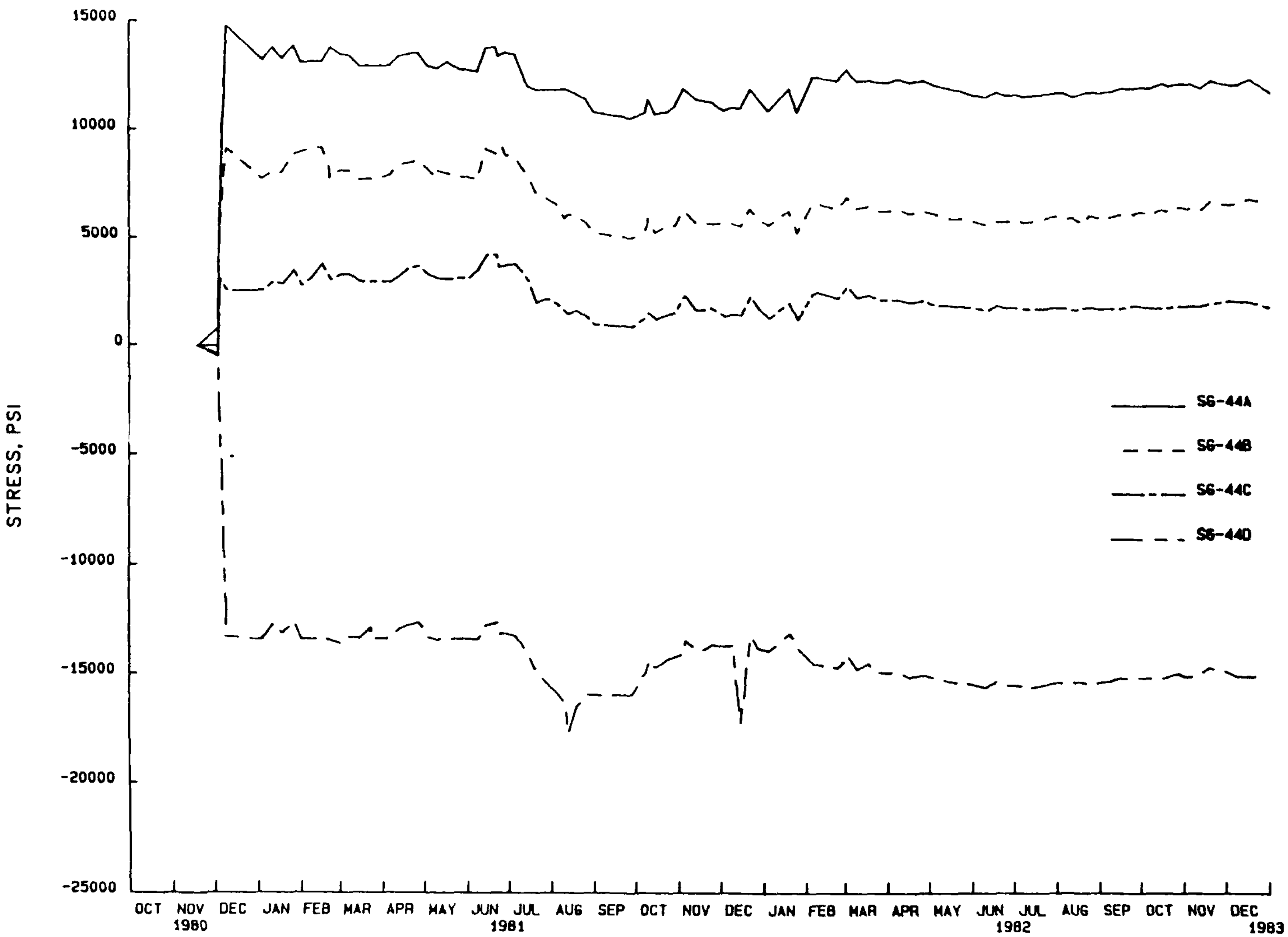


Figure A-40. (B-222) Time history plots of sheet pile strain gauges in dam (SG-44) 1980-1982.

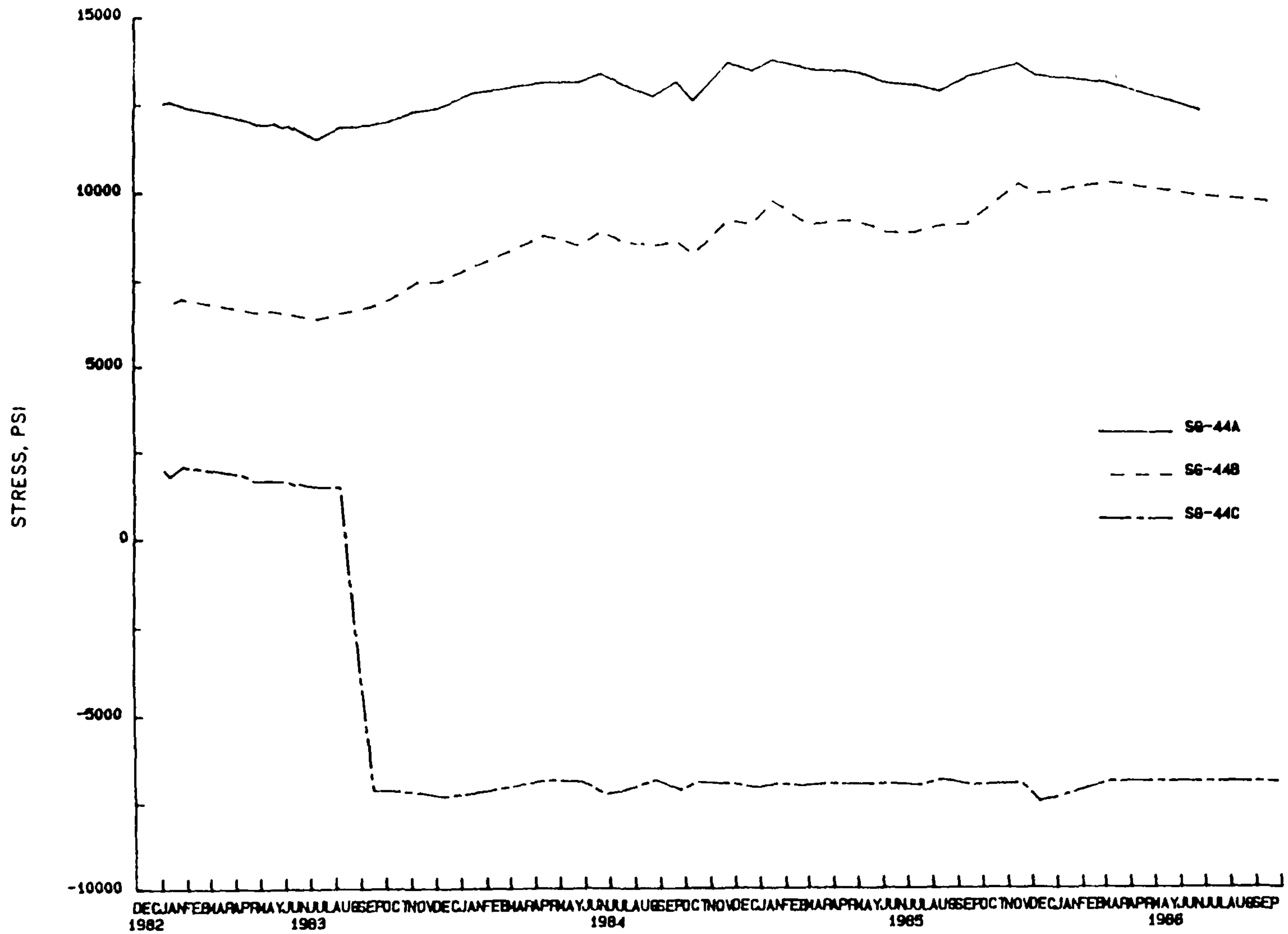


Figure A-41. (B-223) Time history plots of sheet pile strain gauges in dam (SG-44) 1983-1986.

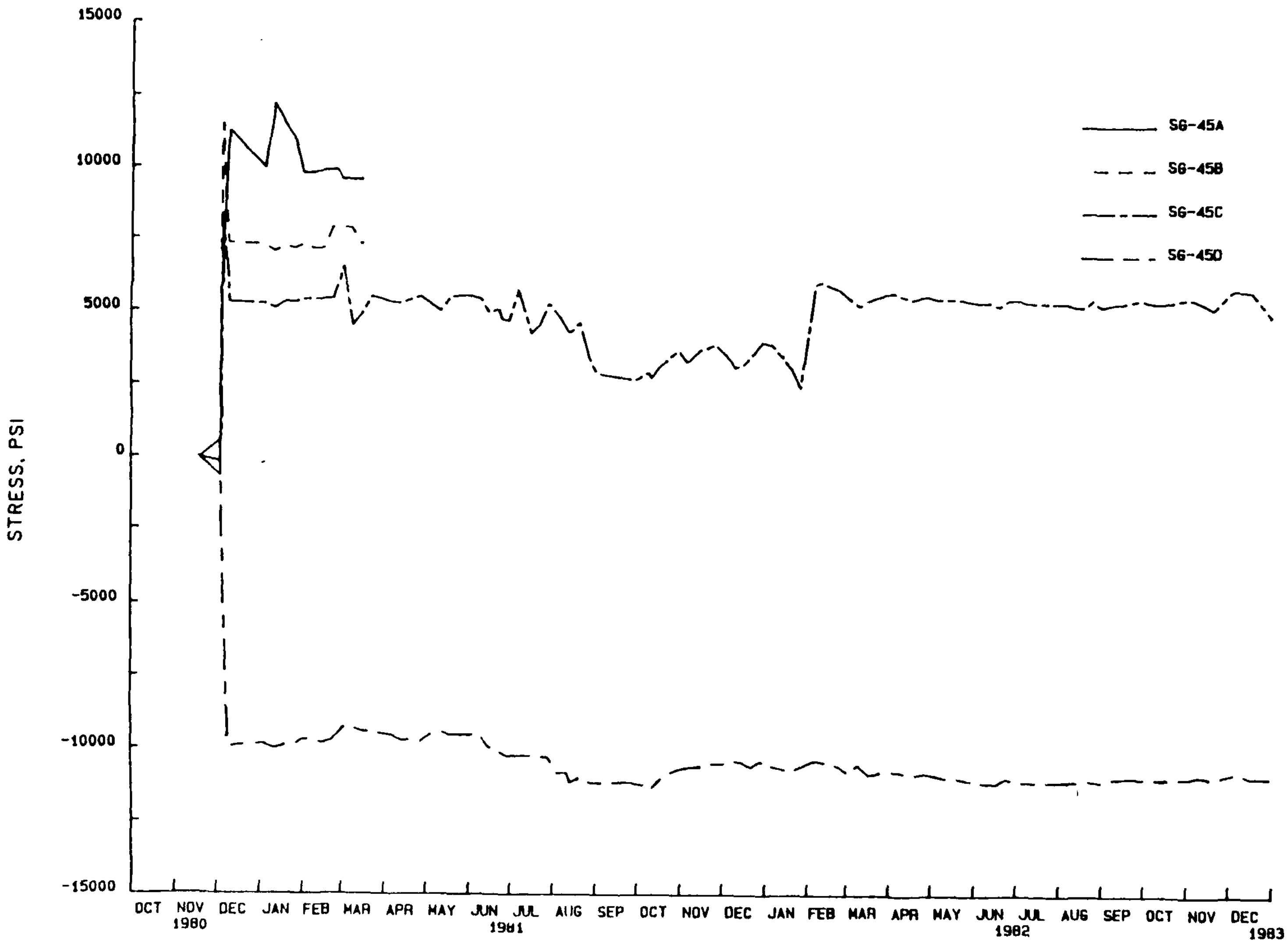


Figure A-42. (B-224) Time history plots of sheet pile strain gauges in dam (SG-45) 1980-1982.

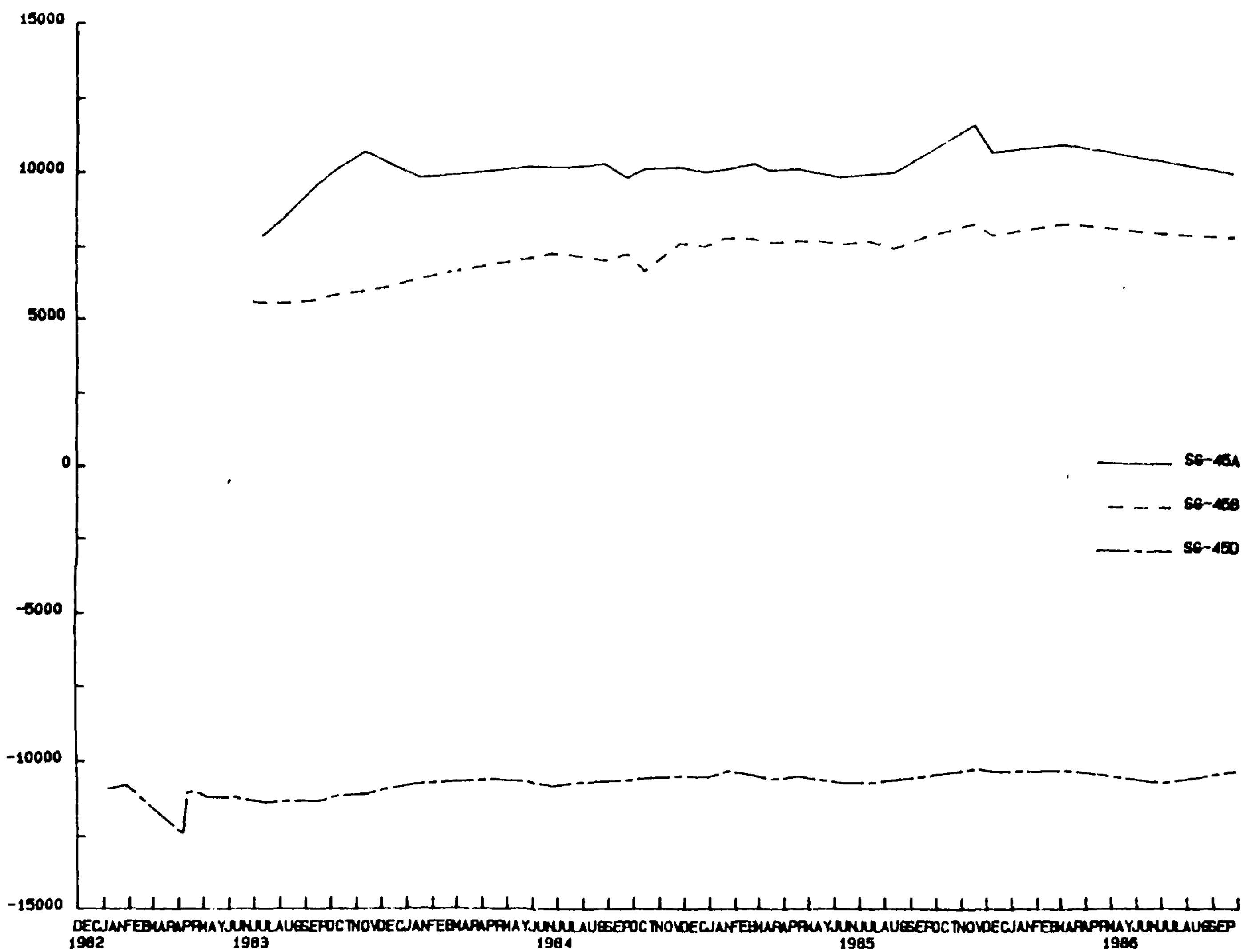


Figure A-43. (B-225) Time history plots of sheet pile strain gauges in dam (SG-45) 1983-1986.

Figure A-44. (B-226) Time history plots of sheet pile strain gauges in dam (SG-46) 1980-1982.

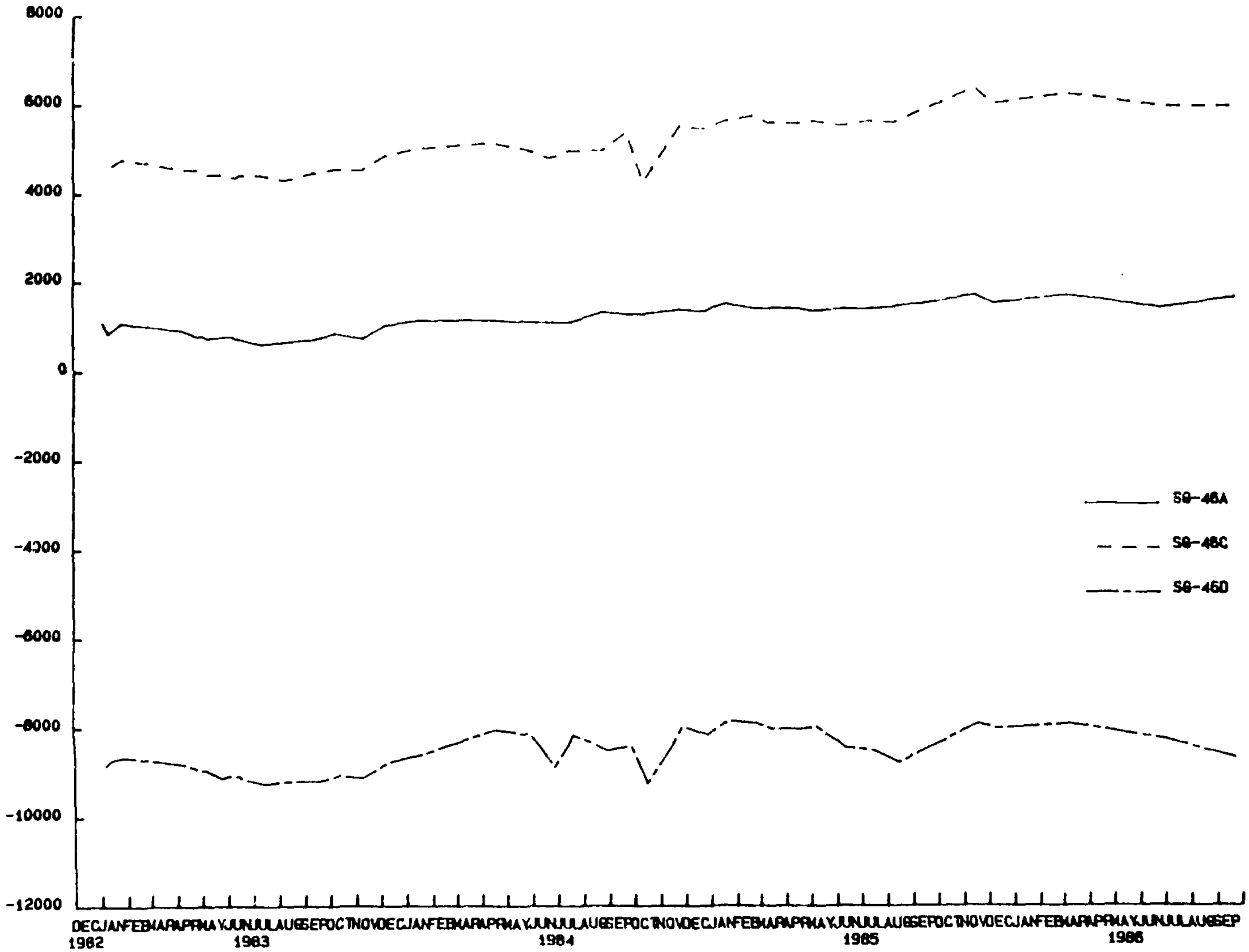


Figure A-45. (B-227) Time history plots of sheet pile strain gauges in dam (SG-46) 1983-1986.